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SHIELD TUNNELS OF THE CHICAGO SUBWAY

BY KARL TERZAGHI, MEMBER*

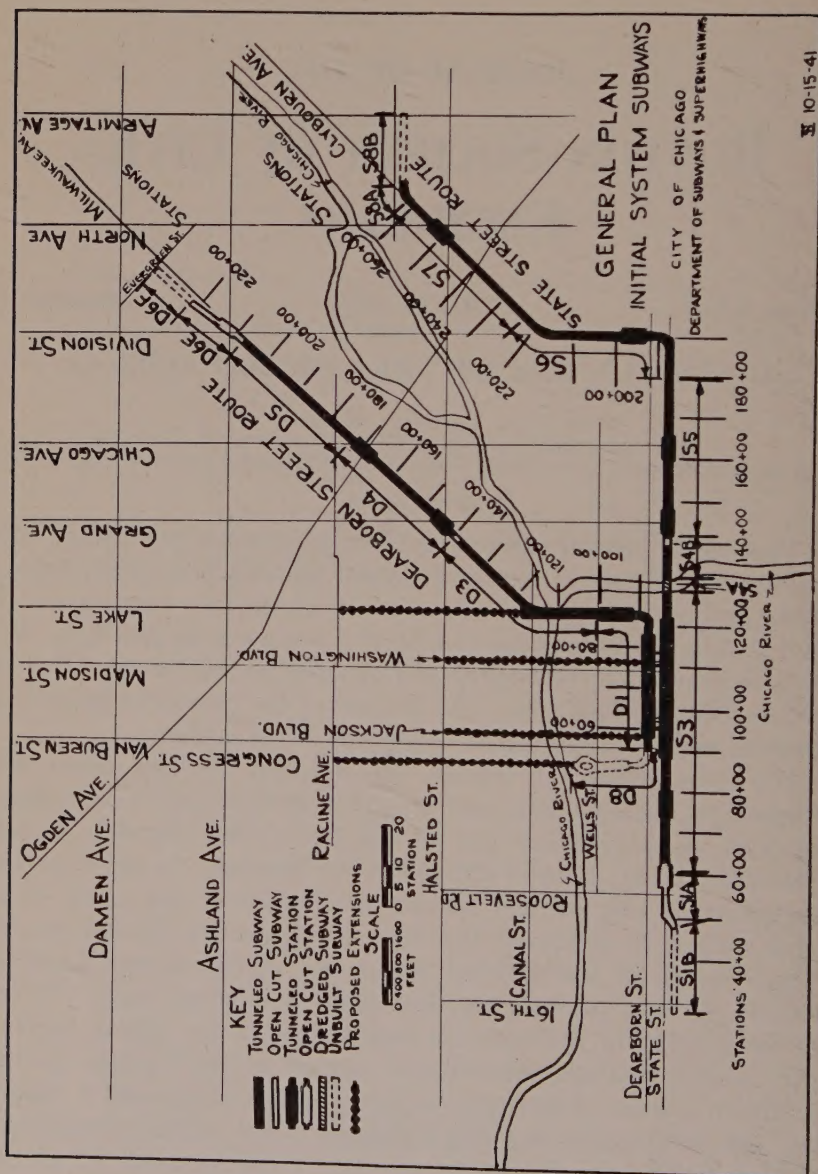
(Presented at a meeting of the Boston Society of Civil Engineers held on May 20, 1942.)

INTRODUCTION

THE initial subway system of Chicago includes two routes, one called the State Street Route and the other the Dearborn Street Route. The location of these routes with reference to the lake shore is shown in Fig. 1. The tunnel sections consist of 7.7 miles of double tubes and station sections. All the tunnels are entirely surrounded by what is known as Chicago blue clay. The bottom of the tunnels is located at a depth of approximately 50 ft. below the street surface.

For constructing the tunnels two different methods came into consideration, the liner plate and the shield method. In the liner plate method the pressure which acts on the roof of the tunnel is carried by steel ribs which transfer the load onto footings. In order to use this method the clay located beneath the bottom of the tunnel must be stiff enough to withstand the downward pressure exerted by the footings. North of the Chicago River this condition was satisfied. Therefore north of the river the liner plate method was used. On the other hand, south of the river the clay was much too soft to support heavily loaded footings. Inadequate support of the footings of the arch ribs in a liner plate tunnel is likely to cause not only a failure of the roof of the tunnel but also a collapse of the adjoining buildings. Therefore in the loop district sound engineering required the use of

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the shield method in spite of the higher cost of construction of this type of tunnel. The considerations which led to this decision and the methods of construction which have been used on the liner plate tunnels were described in another paper.¹ The same paper contains a description of the soil conditions and soil investigations. The following article is devoted only to the shield tunnels.

The shield tunnels comprised two miles of single-tube tunnels, station sections included. They were divided between two contracts, S-3 which followed State Street, and D-1 which constituted part of the Dearborn route. On account of several circumstances the construction of the shield tunnels involved unusual difficulties without any precedent in this field. One of them was due to the fact that the tunnels had to be shoved through a space part of which was occupied by an old freight tunnel lined with concrete. The lining of the tunnel had to be removed ahead of the shield and the interior of the tunnel had to be filled with sand. Therefore part of the diaphragm of the shield was always in contact with the remnants of the concrete lining and the sand fill. A second complication was due to the presence of a layer of stiff clay with a variable thickness at a short distance above the roof of the tunnel. Since the strength and the thickness of this crust have a decisive influence on the heave, the control of the heave required continuous adaptation of the method of construction to the changes in the soil conditions above the roof. A third complication arose from the necessity of constructing the tunnel in the immediate vicinity of sub-basements whose side walls rested on shallow footings. In one instance the shortest distance between the cutting edge of the shield and the curb wall of the adjoining sub-basement was equal to a few feet.

On account of these complications it was obvious that the shield tunnels could not be constructed without affecting to some extent the public utilities located above the tunnels and the adjoining buildings. Hence from the very beginning every effort was made to reduce these effects to the inevitable minimum. To achieve this purpose it was necessary to keep track of the effect of shoving by continuous measurements, to ascertain without delay the causes of every incident and to modify the construction procedure from step to step in accordance

¹K. Terzaghi, Liner-Plate Tunnels on the Chicago (Ill.) Subway. Am. Soc. C.E., Proc. June, 1942.

with the results of the observations. To the author's knowledge, the care with which this was done is without precedent in the field of shield tunneling. Owing to this care, construction was completed without serious accident and the results of the observations provide us with a complete record of the effect of shoving on adjoining structures. The observations during construction included heave and settlement observations on the street surface, in the sub-basements of adjoining buildings and on underground reference points, the measurement of the pressure which acted on rigid obstacles such as walls and footings in the ground during the shoving operations, the measurement of the deformation of the tunnel tube and of the stresses in the lining in an experimental section on contract S-3 and the measurement of the pore water pressure in the clay adjoining the tunnel. The paper contains an abstract of the results of these observations.

SOIL CONDITIONS

Fig. 2 is a section through the soil along contract S-3 (State Street

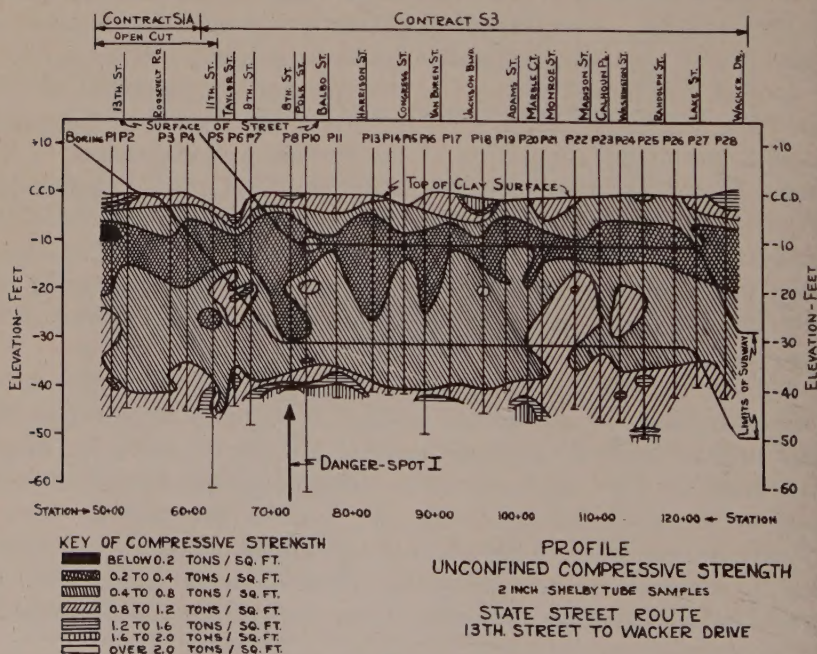


FIG. 2

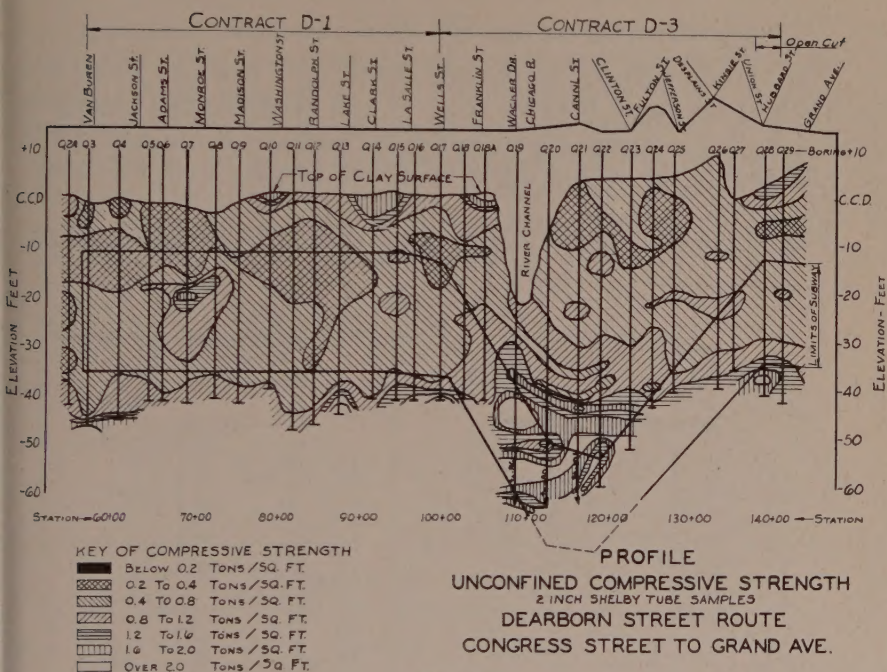


FIG. 3

Route) and Fig. 3 is a similar section along contract D-1 (Dearborn Street). On both contracts the bottom of the tunnel is located on fairly soft clay with an unconfined compressive strength of less than 0.8 tons per sq. ft. and over the major part of the length of the tunnel the roof is located beneath the layer of very soft clay with an unconfined compressive strength of less than 0.4 tons per sq. ft. Within the space to be occupied by the tunnels, the consistency of the clay ranged in an erratic manner between soft and very soft, with isolated lenses and pockets of stiffer material with a compressive strength of more than 1.6 tons per sq. ft. Hence the soil conditions were far from being uniform and the details of the variations were unknown. The surface of the clay is located at a depth of about 15 ft. below the street surface, approximately at the level of the lake. The uppermost part of the clay, to a depth of about 5 ft. below the clay surface, was very much stiffer than the clay located at the elevation of the tunnels. Yet, the thickness and the strength of the top crust were extremely variable.

METHOD OF TUNNEL CONSTRUCTION

The outside diameter of the individual tubes is about 25 ft. and the width of the space between them at mid-height of the tunnels is about 2 ft. 9 in. At any given station the passage of the first shield preceded that of the second shield by a time ranging from two weeks to two months.

Fig. 4 shows the essential parts of the shield. The shield consisted

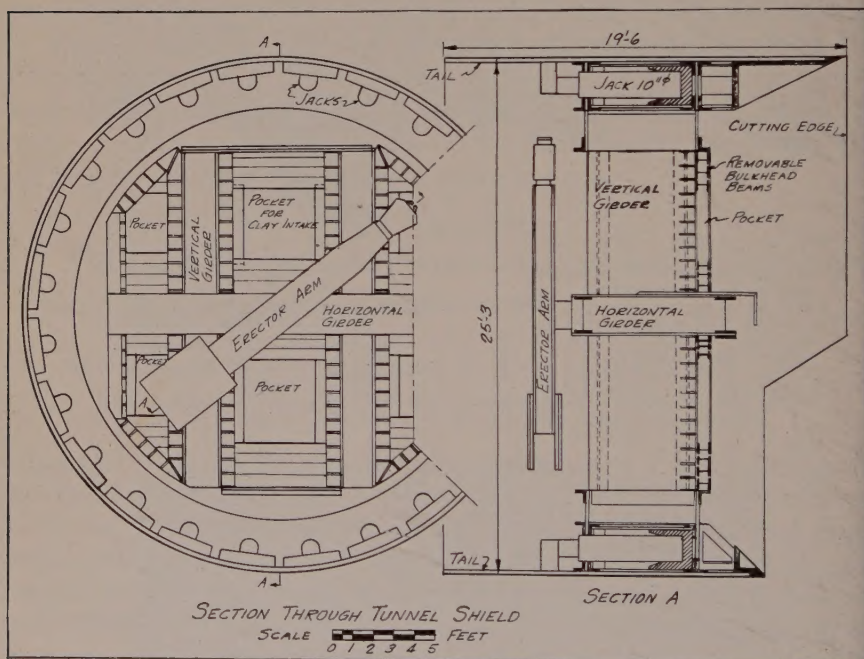
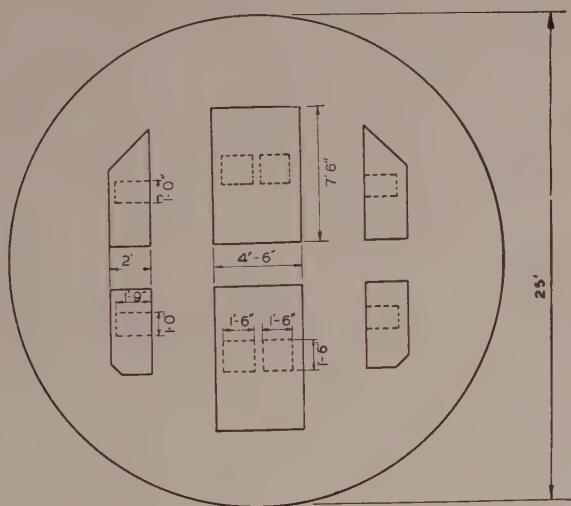


FIG. 4

of a cylindrical tube with a diameter of about 25 ft. and a length of about 20 ft., reinforced by a diaphragm. The diaphragm occupied the position midway between front and rear of the cylindrical section. It served as a support for 24 jacks which were used for advancing the shield. The jacks, each with a capacity of 200 tons, were distributed uniformly around the shield just inside the cylindrical wall. By shoving backward against the lining already in place, these jacks caused the

forward movement of the shield. The diaphragm was stiffened by one horizontal girder at mid-height and two vertical girders which divided the horizontal diameter of the diaphragm into three approximately equal parts. These three girders divided the face of the shield into six compartments in which men could stand and remove the clay from the face. The front face of each compartment contained an opening whose size could be varied within certain limits by the insertion or removal of horizontal bulkhead beams, as indicated in Fig. 4. Fig. 5



Solid lines indicate boundaries of largest openings
Dotted " " " " smallest

FIG. 5.—EXTREME SIZES OF OPENINGS IN THE SHIELD

shows the extreme limits for the size of the openings. The smallest openings represented 3.5% and the largest 20% of the area covered by the shield. During most operations the shield advanced while clay flowed into the tunnel in ribbons which were cut and disposed of by the mining crew. The horizontal and vertical girders also provided the housing for 12 breasting jacks which could be used to support the working face in front of the diaphragm, whenever it was desirable or necessary to mine by hand ahead of the shield. Mounted at the center of the shield at the rear of the horizontal girder was the erector arm,

a hydraulically operated machine which lifted and placed a segment of the temporary lining under the protection of the tail of the shield before each shove.

The lining placed by the erector arm was composed of welded steel segments, as shown in Fig. 6. A standard complete ring of lining

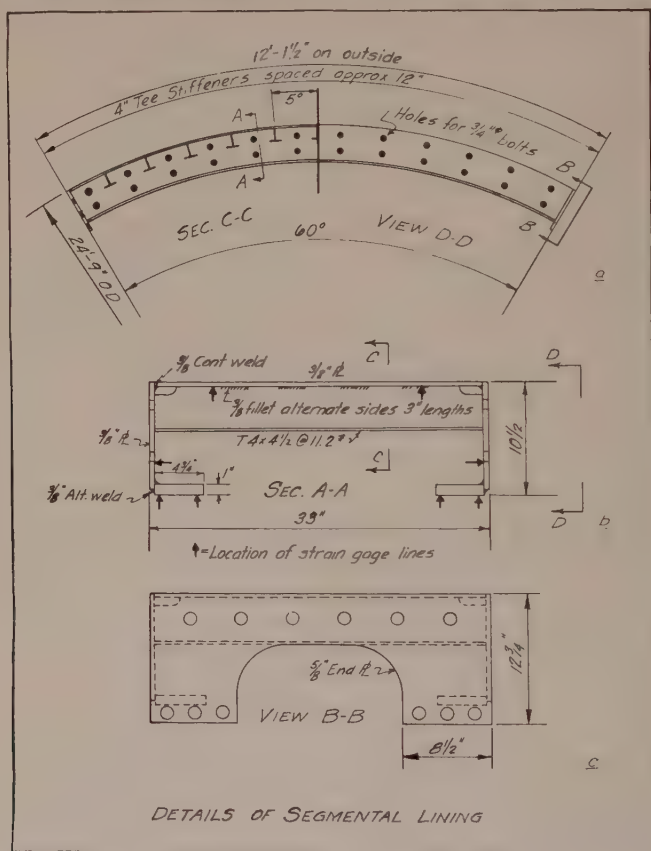


FIG. 6.—TEMPORARY TUNNEL LINING

contained six segments each approximately 12 ft. in length, and a short tapered piece called the key. Variations in the type of lining were made where portions were to be removed later because of station con-

struction. The width of each ring of lining was 2 ft. 9 in., which is equal to the length of the shoves made by the shield.

Mining by means of the shield involved two alternating procedures, making the shove and setting the lining. In making a shove, pressure was applied to all or to some of the jacks and the shield began to move forward. The slow forward motion of the shield was accompanied by a rapid inward squeezing of the clay through each of the six pockets. The rate of shoving was adjusted so that the quantity of clay coming in through each pocket could be handled by the miners. In the two central pockets which were usually kept larger than the side pockets one miner cut off slabs of clay, either by means of a steel wire or by means of a power knife connected to an air tugger. Two muckers were required to receive the slabs and to place them in the muck cars. One miner and one mucker generally were sufficient for the side pockets.

The thickness of the tail of the shield was $2\frac{1}{2}$ in. and a clearance of $\frac{3}{4}$ in. was provided in order to facilitate the erection of the lining. Therefore an annular space $3\frac{1}{4}$ in. wide existed around the temporary lining immediately after the tail piece cleared a given location. In order to fill this void and to prevent settlement, pea gravel was injected through holes in the liner plates beginning at the bottom and working up toward the top. The grouting operations were carried on continuously during shoves. The grout was injected by means of compressed air in a manner similar to that used in the liner-plate tunnels (See Ref. 1).

At the completion of the required length of shove the mining crew retired and the six segments comprising the ring were transported to the shield. The segments were erected starting at the bottom and were bolted on the ends to each other and on one side to the existing lining. In order to prevent the blowing of pea gravel into the shield during grouting operations, it was necessary to place a seal of hemp between each ring and the tail piece of the shield. When the ring was erected, but before the drift pins were replaced by bolts, a new shove was begun by the mining crew while the steel gang continued to tighten the bolts. Under very favorable conditions the length of time for a complete shove, including the erection of the ring, was about 1 hour and 20 minutes. The shove itself required approximately 1 hour and the

erection of the ring 20 minutes. Other incidental operations, however, and ordinary delays lengthened the time required. As an average a rate of 10 shoves per 24 hour day was maintained, thus effecting an average progress of 27 feet of tunnel per day.

Construction of the second tunnel was identical with that of the first. Horizontal struts were placed in the first tube opposite the position of the shield in the second tube in order to prevent excessive deformation of the structure already built. Concrete was not placed in the first tube until some time after the passage of the second shield. In twin tube sections the complete concrete lining was practically circular in shape and was made in two pours. The invert pour included the bottom as well as the curb and the walk way on either side. The arch pour comprised the remainder of the section.

Over a considerable part of the length of the shield tunnels an existing freight tunnel, 8 ft. wide, located about 40 ft. below the center line of the street, interfered with the advancement of the shield. It was necessary to remove the concrete lining of this tunnel before arrival of the shield. Since it was inevitable that the removal of the old freight tunnel should cause some settlement of the street surface, the method of removal received careful attention. After some experimenting, it was found that the following procedure gave the most satisfactory results. The structure was broken out largely by blasting in approximately 15 ft. lengths at a distance several hundred feet ahead of the shield. The demolished part of the freight tunnel was filled with sand.

While shoving the shield, part of the diaphragm of the shield was always in contact with the remnants of the concrete lining and the sand fill. Both materials had to be removed through the pocket next to or within the area of contact, which caused considerable delay. At each street intersection a freight tunnel intersection also occurred. Most of these intersections not only provided a crossing, but three or four turn-outs to enable the freight tunnel trains to turn in any of the four directions. This resulted in a complicated system of intersecting tunnels. It was difficult to break up the lining and to brace walls after the destruction of the lining without causing settlement of the street surface. In these locations it was found necessary to use heavy timbers and to leave them in place while filling the tunnels with sand. The

approaching shield further distorted the remnants of the tunnels. It happened quite often that the advance of the shield was obstructed by large timbers in various positions. Their removal required laborious mining and chopping operations ahead of the shield whereby the progress was sometimes slowed down to three or four shoves per day instead of the normal number.

Considerable difficulties were also encountered in station sections, while removing the body of clay which separated the tubes. However, these operations are beyond the scope of this paper.

INFLUENCE OF THE SIZE OF THE OPENINGS ON THE RESISTANCE AGAINST SHOVING AND THE HEAVE OF THE STREET SURFACE

The shield consists essentially of an almost rigid, circular slab with six openings which is pressed against a vertical face of clay. The diameter of the slab is about 25 ft. It covers an area of about 500 sq. ft. and its center is located at a depth of about 40 ft. below the surface of the street. The force required to press the shield in a horizontal direction into the clay and the movement of the street surface produced by the advancement of the shield depend on the consistency of the clay and on the size of the openings.

As a preliminary to the description of what has been observed in the tunnel and on the street during shoving operations, two fictitious possibilities will be considered, namely the shield is completely closed (percentage openings zero) and the shield is completely open (100 per cent openings).

If the shield is completely closed, the force P required to shove the shield into the clay by means of jacks can be resolved into the following parts:

Force S required to overcome the adhesion between the outside of the shield tube and the clay. The contact area is $25 \times \pi \times 20 = 1600$ sq. ft. and the adhesion is roughly 700 lbs. per sq. ft. Hence $S = 1100$ kips.

Force P_1 required to shove the shield on the assumption that the shearing resistance of the clay is equal to zero. On this assumption the clay would act on the shield, with an area of 500 sq. ft., like a liquid with a unit weight of about 110 lbs. per cu. ft. The center of the shield is located at a depth of about 40 ft. below the surface. Hence $P_1 = 110 \times 500 \times 40 = 2,200,000$ lbs. = 2200 kips.

Force P_2 required to overcome the shearing resistance of the clay on the assumption that the unit weight of the clay is equal to zero. Observations in cuts² indicate that the angle of effective shearing resistance of the Chicago clay is practically equal to zero, unless the forces act on the clay long enough to produce consolidation and the shearing resistance of the clay is roughly equal to one-half of the non-confined compressive strength of undisturbed samples, regardless of the depth at which the samples have been obtained. The average non-confined compressive strength of the soil surrounding the tunnels on the shield tunnel sections is roughly 1200 lbs. per sq. ft. From small scale tests it is known that the resistance per unit of area against penetration of a circular disk into a plastic material is approximately equal to 3 times the non-confined compressive strength of this material, whence $P_2 = 3 \times 500 \times 1200 = 1,800,000$ lbs. = 1800 kips.

The jacks must also overcome the friction F in the jacks themselves which is roughly equal to 25 per cent of the total jack pressure or equal to $0.25 P$. On the other hand, the action of the jacks on the shield is assisted by the air pressure P_a , roughly 13 lbs. per sq. in. or 1800 lbs. per sq. ft. The air pressure acts on an area of 500 sq. ft. or $P_a = 1800 \times 500 = 900,000$ lbs. = 900 kips.

From the equation

$$P = S + P_1 + P_2 + 0.25P - P_a$$

one obtains

$$P = \frac{1}{0.75} (S + P_1 + P_2 - P_a) = 5600 \text{ kips}$$

An advancing shield without openings would displace the clay without reducing its volume. Therefore the space through which the street surface would heave would be equal to the volume of the tunnel, or 500 cu. ft. per foot of length. The heave would produce a ridge with a width roughly equal to twice the diameter of the tunnel. Its height would be equal to about 15 ft.

As a second, extreme possibility, we assume that the shield is entirely open. In this case the force required to shove the shield would be equal to

²R. B. Peck, Earth Pressure Measurements in Open Cuts, Chicago (Ill.) Subway. Proc. Am. Soc. C.E., June, 1942.

$$P = S + 0.25P$$

$$\text{or} \quad P = \frac{1}{0.75} S = 1500 \text{ kips}$$

The air pressure P_a acts directly on the clay. If the clay is very soft and the air pressure low, the clay would flow freely into the tunnel and a deep, troughlike depression would appear on the surface above the tunnel. On the other hand, if the clay is fairly stiff and the air pressure is adequate, it would be necessary to excavate the clay. Nevertheless, the surface above the tunnel would subside on account of the plastic yield of the clay towards the working face. (See Ref. 1.) Under unfavorable conditions the settlement could exceed two feet.

In shield tunnel practice the shield is neither completely closed nor completely open. If the shield is completely closed, the amount of heave is determined by the volume occupied by the tunnel. In the following discussions this heave will be called 100%-heave. On the Chicago tunnels the 100%-heave would be equal to about 15 ft. If the shield were entirely open a trough-like depression would appear on the surface above the tunnel. The depth of this trough would depend on the softness of the clay and on the air pressure. In other words the heave would be negative. Hence, if the shield is provided with openings the percentage heave should have a value intermediate between +100% and the extreme negative value. It appears logical to assume that the percentage heave should be a simple function of the percentage openings and it should be possible to represent this relation by a curve similar to the curve C_0 in Fig. 7a. Thus for instance on the Lincoln (formerly Midtown Hudson) Tunnel³ the percentage opening was equal to 0.5% and the corresponding heave was approximately 70%. When shoving a shield beneath a street in a city it is desirable to keep the heave as close to zero as possible. The shape of the curve which represents the relation between percentage opening and heave and the abscissa of its point of intersection with the horizontal axis depend on the character of the clay. Hence when a shield is shoved through a mass of clay with variable properties, it should be possible to obtain satisfactory results by adapting the size of the openings to the soil

³G. M. Rapp and A. H. Baker, The Measurement of Soil Pressures on the Lining of the Midtown Hudson Tunnel. Proc. Intern. Conf. on Soil Mechanics, Cambridge, Mass., 1936. Vol. II.

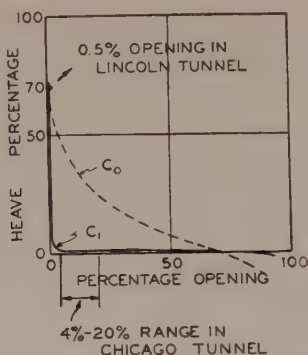


FIG. 7.—(a) INFLUENCE OF PERCENTAGE OPENING ON PERCENTAGE HEAVE

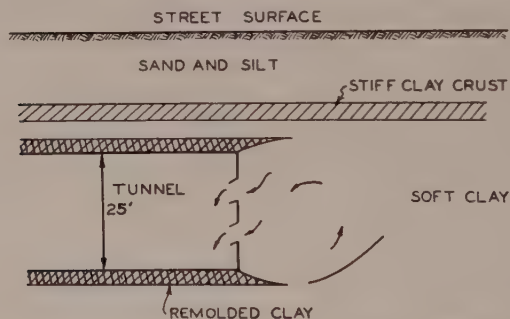


FIG. 7.—(b) FLOW OF CLAY TOWARD SHIELD OPENINGS

conditions. One should also expect that the rate of shoving should have an appreciable influence on the heave. By increasing the rate one creates a viscous resistance against a plastic deformation of the clay which in turn would increase the heave. Thus the contractor would have two independent means at his disposal for regulating the heave, namely, a change of the percentage openings and a change of the rate of shoving.

However, on the subway of Chicago, experience did not confirm either of the preceding conclusions. Neither the percentage opening nor the rate of shoving had an appreciable influence on the heave, and by plotting percentage heave against percentage openings as accurately as possible, one obtained within the range of observation, 4 to 20 per

cent openings, an almost horizontal line C_1 , located very close to the horizontal axis. (Fig. 7a.) The reason for the discrepancy between theory and reality is readily seen. The preceding conclusions were based on the assumption that the shield is shoved through homogeneous, soft clay. In reality, the roof of the tunnels is located beneath a stiff and relatively rigid crust of clay which separates the soft clay from the superimposed stratum of sand and silt, as shown in Figs. 2 and 3. Fig. 7b is a simplified section through this crust and the center line of the tunnel. On account of the existence of this crust the heave of the street surface is unimportant unless the pressure exerted by the shield is great enough to break the crust. If the shield is completely closed, the crust breaks and the heave is 100%, but the pressure required to produce this failure is very high. Hence, if there are openings in the shield at all, the pressure required to force the soft clay through the openings into the tunnel is smaller than the pressure required to break the crust, though the openings may be small. Shoving without breaking the crust produces not more than a gentle upward deflection of the crust, comparable to the upward deflection of the top of a pressure cell due to an internal pressure. The radical, plastic deformations of the clay are limited to the masses located in the path of the shield. The upward deflection of the crust depends primarily on the thickness and the stiffness of the crust, which are purely accidental factors, unknown to the operator of the shield. The influence of these factors on the heave is undoubtedly more important than the influence of a variation in the size of the openings between the limits of 4% and 20%. Therefore it is not surprising that the heave in Chicago appeared to be fairly independent of the size of the openings. The presence of the crust explains much of what has been observed in the shield tunnels in Chicago. It also eliminates the possibility of applying the results of the observations to shield tunnels in homogeneous materials. The following section contains a brief description of the phenomena produced by shoving the shield in Chicago and an interpretation of the findings.

RESISTANCE AGAINST SHOVING AND THE MOVEMENT OF THE STREET SURFACE

During the construction of the shield tunnels complete records

were kept of the shoving pressure, the number and location of jacks in operation, the size of the openings through which the clay entered the tunnel, the air pressure and the rate at which the shield advanced. The air pressure was fairly constant and roughly equal to 13 lbs. per sq. in. Settlement observations were carried out at several points on cross sections located 10 ft. apart in the direction of the tunnel. By means of telephone communication with the heading, shoving operations were controlled so that whenever the heave of the street surface exceeded a value of about 4 in., shoving operations were reduced or entirely stopped until the heave was within permissible limits. If the slowing-down process did not remedy the situation, it was necessary to mine the clay in front of the shield, until the obstructions were removed.

The combined capacity of the jacks was $24 \times 400 = 9600$ kips. The pressure required to shove the shield averaged 5000 kips. It is somewhat smaller than the value $P = 5500$ kips which has been obtained by computation in the preceding section for the force required to shove a completely closed shield.

The reaction on the bearing surface of the jacks was transmitted through skin friction from the tunnel lining onto the adjoining clay. Assuming that the active part of the skin friction increased from zero at the end of the section of pressure transmission with a length L in simple proportion to the distance from this end, to a maximum value of 700 lbs. per sq. ft. in the proximity of the shield, the length L is determined by the equation

$$L \times 25\pi \times \frac{1}{2} 700 = 5,000,000$$

whence

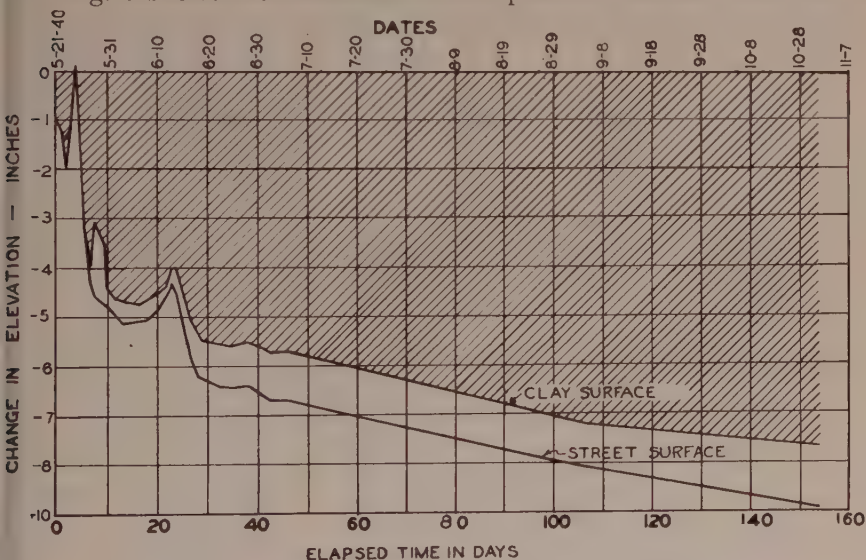
$$L = 180 \text{ ft.}$$

In reality L is somewhat greater. Measurable compressive strains in a longitudinal direction (parallel to the center line of the tunnel) were found to exist up to a distance of 250 ft. from the shield and relative displacements between the rings were detected up to a distance of 300 ft. from the shield. Since the rings were not in perfect contact with each other, the strain readings were very erratic. Therefore it was not possible to ascertain the distribution of the longitudinal stresses over the section of pressure transmission.

The movement of the street surface can be divided into the following successive stages:

1. Settlement due to demolishing the freight tunnel.
2. Heave due to shoving the first shield.
3. Settlement due to the stretching of the clay located immediately behind the shield.
4. Heave during the passage of the second shield, and
5. Progressive settlement of the street surface after the passage of the second shield. This last stage can only be accounted for by a process of consolidation.

Fig. 8 shows the movement of one point of the west shoulder on



SETTLEMENT OF STREET SURFACE AND CLAY SURFACE
CONTRACT D-1 STA. 83+70

FIG. 8

the street surface (lower curve) and of one point on the clay surface beneath the west shoulder on contract D-1 at Sta. 83 + 70. It should be noticed that the distance between the street and the clay surface decreased during the first weeks after construction started. This phenomenon also occurred but to a lesser extent above the liner-plate tunnels. It indicated a slight compaction of the top stratum. It was most likely due to the pounding of the street traffic combined with the

change in the state of stress in the top stratum produced by the mining operations at a deeper level.

The settlement due to the removal of the freight tunnel (first stage of movement) ranged between about $\frac{1}{2}$ in. and 2 in. At the street intersections it was greater because these points were located above freight tunnel intersections.

As soon as the shield approached a cross section, the street surface rose, as shown by curve *A* in Fig. 9. The heave produced a flat ridge

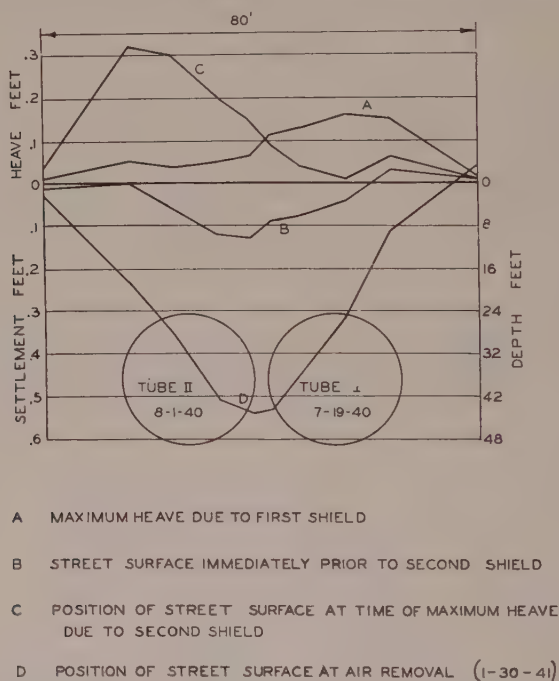


FIG. 9.—SUCCESSIVE STAGES IN MOVEMENT OF STREET SURFACE ABOVE SHIELD TUNNELS

whose width was approximately equal to twice the diameter of the tunnel. The height of the ridge ranged between about 1 in. and 4 in. The corresponding percentage heave ranges between 0.6% and 2.4% which is negligible. At the street intersections it was likely to be greater than 4 in. Originally it was believed that the greater heave at street intersections was chiefly due to the effort to compensate for the

settlement produced by the removal of the freight tunnels at these points. However, subsequent experience has shown that the possibilities for controlling the heave by means other than mining ahead were very limited as explained in the preceding section. Hence it is more likely that the greater heave at street intersections was chiefly due to severe injury to the stiff crust during the preceding removal of the system of intersecting freight tunnels. A weakening of the top crust is necessarily associated with an increase of the heave.

Fig. 10 shows a vertical section through the center line of the tunnel on contract D-1 between Sta. 68 and 80. The ordinates of

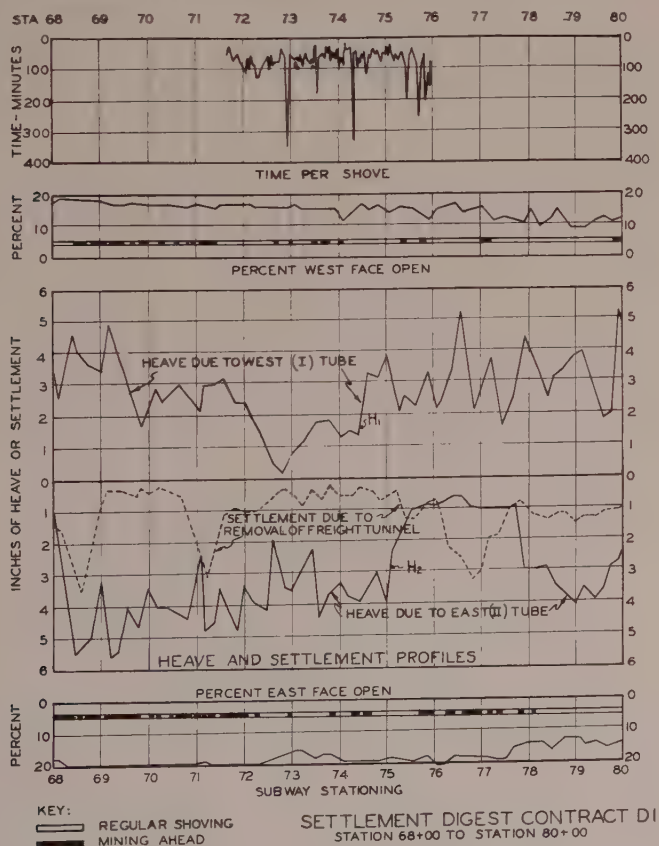


FIG. 10

curve H_1 represent the maximum heave of the street surface during the passage of the first shield with reference to the deepest position of the point of observation prior to the approach of the first shield. The ordinates of the curve H_2 (plotted downwards) indicate the maximum heave during the passage of the second shield with reference to the deepest position of the point of observation before the second shield approached the profile. The percentage opening in the first shield was approximately 17% and that in the second shield about 20%. The rate at which the shield was advanced was at an average equal to 80 minutes per shove (2 ft. 9 in.). On the figure are also indicated those sections where it was necessary to mine the clay in advance of the shield.

Both the settlement of the street surface above liner plate tunnels and the heave above shield tunnels depends to a large extent on the strength and the thickness of the stiff crust located above the tunnel. Therefore both the settlement and the heave profiles should reflect the mechanical properties in the same manner. However, when comparing a settlement profile for a liner plate tunnel (for instance, Fig. 18 in Ref. 1) with the heave profile in Fig. 10, one can see that the heave is far more variable than the settlement, although every effort was made on the shield tunnels to maintain the surface of the street as closely as possible at its original elevation. In order to understand the causes of the erratic and apparently unmotivated sequence of small and great heave, it is necessary to consider the deformation of the clay adjoining the shield.

When watching the clay coming out of the openings, one got quite decidedly the impression that the clay flowed towards the openings from above, as indicated by arrows in Fig. 7*b*. Yet the surface of the street moved up and not down. Therefore the loss of ground due to the downward flow of the clay must have been associated with an upward flow at a greater distance from the shield. This conclusion is corroborated by the following observation. Before the approach of the shield the freight tunnel space was filled with heavy timber bracing and sand. The advancing shield kneaded the clay, the sand and the bracing into a non-homogeneous mixture. Originally all the timbers were located below the level of the center of the shield. Nevertheless, a large number of timbers entered the tunnel through the upper open-

ings which indicated that the timbers were dragged in an upward direction, as indicated by an arrow in Fig. 7*b*, before they descended towards the openings.

It has also often been observed that the clay flowing through the ports into the tunnel was uniform for a short period of time and then it changed its character abruptly for another short interval. No such abrupt changes in a horizontal direction have ever been encountered in the undisturbed clay bed. Therefore one is obliged to assume that they represent one of the consequences of the kneading action produced by the shield. If the shield is forced into the clay, the soft portions of the clay flow more readily towards the ports than the stiff ones. Therefore the stiffer chunks and the timbers accumulate in front of the shield until the entire space adjoining one or the other of the openings is entirely occupied by stiff chunks. After that none but stiff material comes out of this opening until the passage for soft material is free again. Occasionally the resistance offered by the stiff chunks was so great that it was necessary to excavate them by hand in front of the shield. The process of accumulation and subsequent discharge of stiff chunks in front of the shield involves periodic change between small and large resistance against the displacement of the clay and leads necessarily to the irregular wave pattern of heave shown in Fig. 10.

After it was understood that the effect of the size of the openings on the heave is considerably less important than the corresponding effect of the periodic accumulation of stiff material in the front of the shield, the attempt to control the heave by changing the size of the openings was abandoned. Under normal operating conditions, the size of the openings was kept constant unless the contractor wanted to change the quantity of clay which entered the tunnel in order to adapt this quantity to the handling capacity of the miners.

The heave produced by the passage of the first shield was followed by an appreciable settlement as shown in Fig. 9. The amount of settlement was usually much greater than that of the preceding heave. A subsidence of 3 or 4 inches was quite common. It could be explained by the clay invading the empty space, $3\frac{1}{4}$ in. wide, which formed behind the tail end of the advancing shield, by incipient consolidation of the layer of completely remolded clay which surrounded the lining

of the tunnel, or by the stretching of the clay adjoining the boundary between the tail end of the advancing shield and the lining of the tunnel. In order to find out which one of the explanations is correct, various observations have been made.

By measuring the rate at which the clay invaded the annular space behind the tail end of the advancing shield before the tunnel was filled with compressed air, it was found that the width of the space decreased at a rate of about one inch per hour. The presence of compressed air undoubtedly reduced this rate to less than half an inch per hour. Furthermore, the grouting operations were conducted such that the time during which the clay could advance into the annular space nowhere exceeded half an hour. Once the space is filled with pea gravel, the clay does not enter the voids of the gravel. This was demonstrated by laboratory tests. Hence the annular space which was formed behind the advancing shield was not responsible for the settlement. This conclusion is in accordance with the observation that the volume of grout, which was injected into the annular space was always almost exactly, and in a few cases even somewhat greater than the initial volume of the annular space. These observations indicate that the settlement cannot be due to an invasion of the annular space by clay. Therefore we must concentrate our attention on the two other possible causes, the consolidation due to remolding, and the stretching of the clay.

In order to get reliable information concerning these two processes three underground reference points have been established on Contract S-3, at Sta. 117 + 40. The reference points were located on a vertical line, 5 ft. off the center line of the tunnel, as shown in Fig. 11. The cutting edge of the first shield passed the station at a time $t = 6$ days. At that time the distance between the reference points was still unchanged. During the following two days the distance $AB = 2.5$ ft. decreased by 0.06 ft. or 2.4% and the distance $BC = 2.8$ ft. by 0.14 ft. or 5%. After these two days the distance AC remained practically unchanged. Yet the settlement of the street surface continued, though at a decreasing rate. Therefore the settlement which followed the first heave was not only caused by stretching, but it was quite obviously associated with and followed by a settlement due to consolidation of the remolded layer. It must be considered

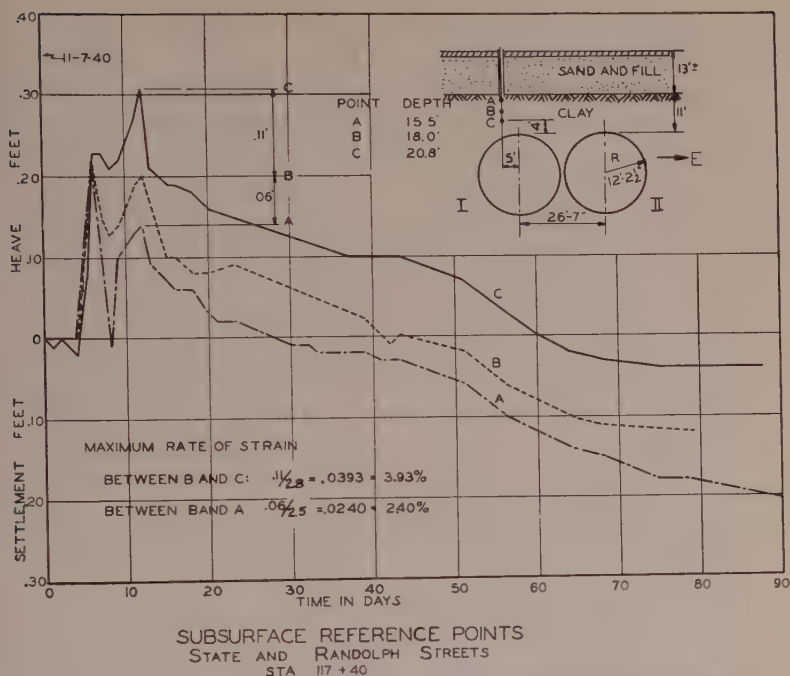


FIG. 11.—MOVEMENT OF SUBSURFACE REFERENCE POINTS ABOVE TUNNEL

inevitable because one cannot shove a shield without stretching and remolding the surrounding clay.

THE SECOND HEAVE AND THE SETTLEMENT OF THE STREET SURFACE AFTER THE PASSAGE OF THE SECOND SHIELD

The second heave, indicated by a second rise in the time-settlement curve shown in Fig. 8, started as soon as the cutting edge of the second shield arrived at a distance of about 20 ft. from the station and it was a maximum immediately after the cutting edge of the second shield passed the station. The second heave constitutes the fourth stage in the movement of the street surface. In order to protect the lining of the first tunnel during the passage of the second shield, horizontal struts were inserted in the first tube at mid-height of the tunnel and they were left in place until the second shield was well beyond the station. Fig. 12 shows the relation between the distance of the shield

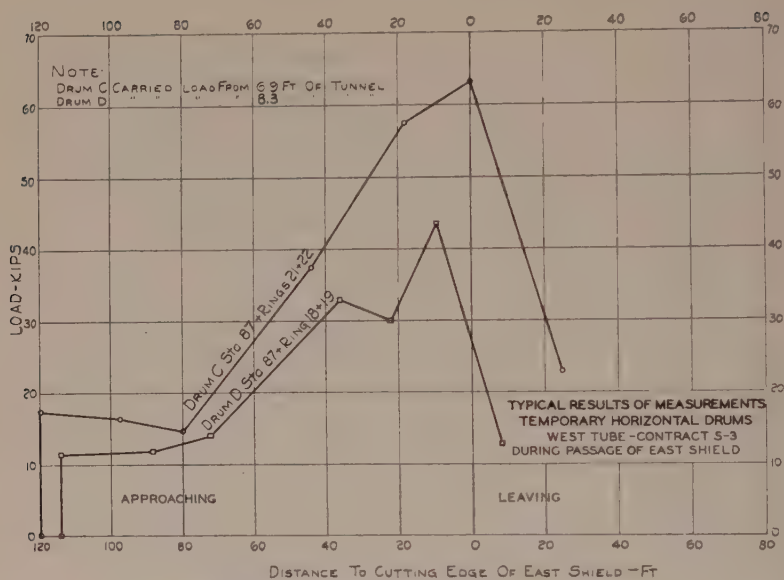


FIG. 12.—PRESSURE IN TEMPORARY STRUTS

from the strut measured in the direction of the tunnel and the pressure in the strut. The greatest measured pressure in the strut was smaller than 10 kips per foot of the length of the tunnel. This pressure is extremely small compared to the pressure exerted by the jacks on the clay in front of the second shield (10 kips per sq. ft.). Hence we are obliged to assume that the major part of the lateral pressure acting on the first tube was transferred directly onto the clay by the ring action of the lining. The lateral displacement of the first tube caused by the passage of the second shield did not exceed 0.5 in.

The second heave was followed by a progressive settlement of the street surface as shown in Fig. 8 (fifth stage). During this process the distance AC in Fig. 11 remained unchanged, which indicates that the thickness of the remolded layer which represented the seat of the settlement due to consolidation was smaller than 4 ft. Beyond the outer boundary of the remolded layer the strain in the clay produced by shoving the shield was rather insignificant. It is a well established fact in soil mechanics that the remolding of a mass of clay initiates a new cycle of consolidation, although the external forces which act on the

clay may remain unchanged. Hence the progressive settlement disclosed by Fig. 8 is an inevitable consequence of the physical properties of clays in general.

The remolded layer of clay surrounds the lining of the tunnel like a thin shell. If a homogeneous shell of this type consolidates, the tunnel should settle through a distance which is roughly equal to one-half of the settlement of the clay surface, although the construction of the tunnel reduced the pressure on the clay located beneath the bottom of the tunnel. If the clay located beneath the bottom of the tunnel is stiffer than the clay above the roof, the settlement of the tunnel should be somewhat smaller than one-half of the settlement of the clay surface. This conclusion is confirmed by the data shown in Fig. 13. The

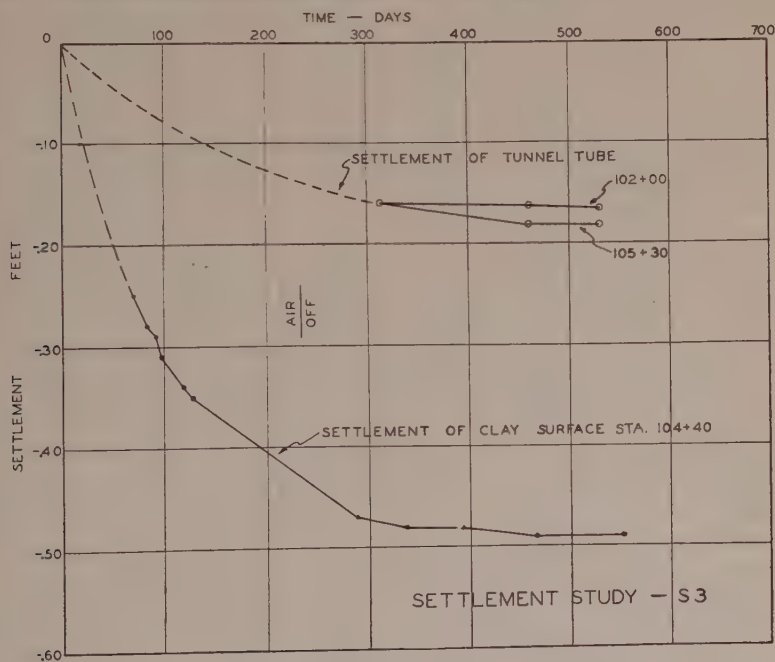


FIG. 13

lower curve shows the relation between the time and the settlement of the clay surface at Sta. 104 + 40 on Contract S-3. The upper curves represent the corresponding relation for two points located on the

invert of the tunnel close to Sta. 104 + 40. The clay located beneath the tunnel was appreciably stiffer than the clay located above the roof. The figure shows that the settlement of the tunnel was at any time roughly equal to one-third of the settlement of the clay surface. The balance of the settlement which was equal to more than 2 in. occurred within a distance of less than 4 ft. from the roof of the tunnel, because the thickness of the layer of clay located at a higher elevation remained unchanged as shown in Fig. 11. In a similar manner the remolded layer located on both sides of the tunnel became thinner. The vacancy was filled by a lateral yield of the undisturbed mass of clay located beyond the outer boundary of this layer towards the tunnel, which in turn caused a settlement of the clay surface on both sides of the strip occupied by the tunnels.

Every process of consolidation involves a flow of water out of the consolidating clay. The direction of drainage depends on the pore water pressure along the boundaries of the consolidating layer and on the permeability of the materials located beyond these boundaries. In connection with the process of drainage one must distinguish between two successive stages, separated from each other by the elimination of the air pressure in the tunnel.

In the undisturbed clay, prior to the construction of the tunnel, the hydrostatic head at the level of the crown of the tunnel is approximately equal to 16 ft. and at the level of the bottom it is about 41 ft. After the tunnel is mined, the tunnel is filled with air under a pressure of about 13 lb. per sq. in., which represents the equivalent of a head of water about 30 ft. Hence at the crown the air pressure is greater and at the bottom it is smaller than the initial head. The tunnel lining is surrounded with a layer of pea gravel, $3\frac{1}{4}$ in. thick, which in turn is surrounded by a layer of completely remolded clay, several feet thick. Immediately after a clay is remolded, all the forces which act on the remolded clay are carried by an excess hydrostatic pressure in the pore water of the clay. The observations in the experimental section on Contract S-3 have shown that the tunnel is acted upon by an almost uniformly distributed all-around pressure. The intensity of this pressure is roughly equal to the vertical unit pressure p on a horizontal section through the clay at mid-height of the tunnel, $p = 40 \text{ ft.} \times 110 \text{ lb. per cu. ft.} = 4400 \text{ lb./sq. ft.}$ At the boundary between the pea gravel

and the remolded clay, the pore water pressure u_w is equal to the air pressure, which is about 1800 lb./sq. ft. The remolded clay is surrounded by almost undisturbed clay which is practically impermeable. Therefore the major part of the excess water will drain out of the remolded clay into the pea gravel and into the voids of the sand which occupies part of the old freight tunnel. This is the first stage of consolidation. During this first stage the hydrostatic pressure in the pore water of the undisturbed clay is at least as great as it was before the construction of the tunnel because the tunnel is filled with compressed air.

The second stage begins as soon as the compressed air is removed. The removal of the air reduces the pore water pressure at the boundary between the remolded clay and the pea gravel above the free water level in the pea gravel from 1800 lb./sq. ft. to zero, which involves a simultaneous increase of the consolidation pressure on the remolded clay by 1800 lb./sq. ft. The initial consolidation pressure was $4400 - 1800 = 2600$ lb./sq. ft. In spite of the increase of the consolidation pressure from 2600 to 4400 lb./sq. ft., the effect of the removal of the air pressure on the trend of the settlement curve is hardly noticeable. This is due to the physical properties of remolded clays in general. The compression due to the increase of the consolidation pressure on any remolded clay from 2600 to 4400 lb./sq. ft. is very small compared to the compression produced by the increase of this pressure from zero to 2600 lb./sq. ft.

The removal of the air pressure changes completely the hydrostatic conditions not only in the remolded but also in the adjoining undisturbed clay. In order to get some information on these conditions, six water-pressure gages were installed in the clay adjoining the experimental section at St. 87 + 70 on Contract S-3. The location of the points of observation with reference to the tube are shown in Fig. 14. The first readings were made on June 1, 1941, about seven months after the removal of the air pressure. The figure indicates quite plainly the existence of drainage out of the undisturbed clay through the remolded layer towards the tunnel. Yet the compression of the clay due to the increase of the effective pressure associated with the drainage of the undisturbed clay must have been quite insignificant. Otherwise the removal of the air pressure should have caused a considerable

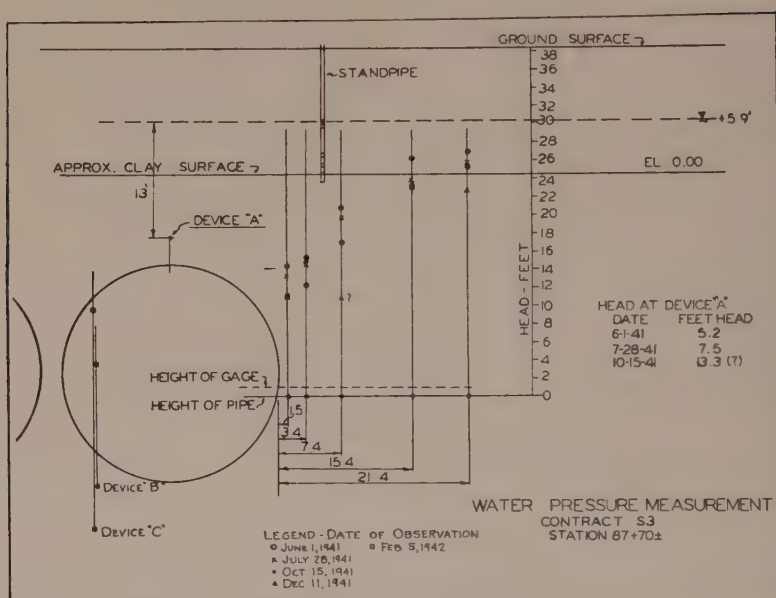


FIG. 14

increase of the rate of settlement of the street surface. Yet no appreciable increase occurred.

All the observations which have been made to date indicate drainage towards the tunnel. However, the number of points of observations and of readings are not sufficient for reconstructing the different phases of drainage. From what is shown in Fig. 14 one can only conclude that the process is by no means simple and that the tunnel acts at least temporarily as a drain. The conditions of final hydraulic equilibrium are not yet known.

EFFECT OF SHIELD SHOVING ON BUILDINGS

The overall width of the two tunnel tubes, including the space between them, was about 52 ft. The center line of the space occupied by the tunnel tubes commonly coincided with the center line of the streets. The width of the streets between building lines was 80 ft. on Lake and Dearborn Streets on Contract D-1, 100 ft. on Contract S-3 south of Madison Street, and 120 ft. on Contract S-3 north of Madison

Street. The heave due to shoving the shield seldom extended beyond the building lines. The maximum heave of any structure (a two-story building on spread footings on an 80-ft. street) at the building line was $\frac{5}{8}$ in. In most other cases it was less than the tolerable error in leveling ($\frac{1}{8}$ in.).

Most of the buildings located in the proximity of the shield tunnels were provided with sidewalk sub-basements. The width of these sub-basement vaults ranged between about 10 ft. on the 80-ft. streets and 16 ft. on the 120-ft. street. The depth of the one-story vaults ranged between 10 and 15 ft. However, some of them were multi-storied. The side walls of the vaults usually consisted of rubble walls resting on continuous masonry footings. The crest of these walls carried the curbstones. The floor of the vaults consisted of a layer of plain concrete between 4 and 6 in. thick, which rested directly on the soil. The ceiling was carried by sidewalk beams which spanned the space between the curb wall and the main wall of the building.

Some of the more modern structures possessed curb walls of reinforced concrete. In these structures the sidewalk beams were encased in concrete and at times the basement floor was reinforced. In a few instances the curb wall rested on piles or caissons, but the continuous footings prevailed regardless of the type of foundation beneath the building proper. The buildings themselves rested upon various types of foundations, including spread footings, piles, caissons to hardpan and caissons to rock. Many of the newer structures are provided with several sub-basements some of which extend as far as the curb line.

Wherever the shield was shoved in the vicinity of a sub-sidewalk vault the shoving operations tended to cause a heave of the floor of the vault. Fig. 15 shows two simplified sections through one of the tunnel tubes and an adjoining vault beneath a street with a width of 80 ft. and 120 ft. respectively. The stiff crust of clay is indicated by a shaded strip. Above the roof of the tunnel the stiff crust carries an overburden consisting of the weight of a layer of sand and silt, about 15 ft. thick. The weight of this layer combined with the stiffness of the crust prevents an excessive heave of the street surface. On the other hand, at the site of the vault the rise of the floor is resisted only by the stiffness of the crust and of the floor. Therefore the passage of the shield causes the floor of the vault to rise unless the crust and

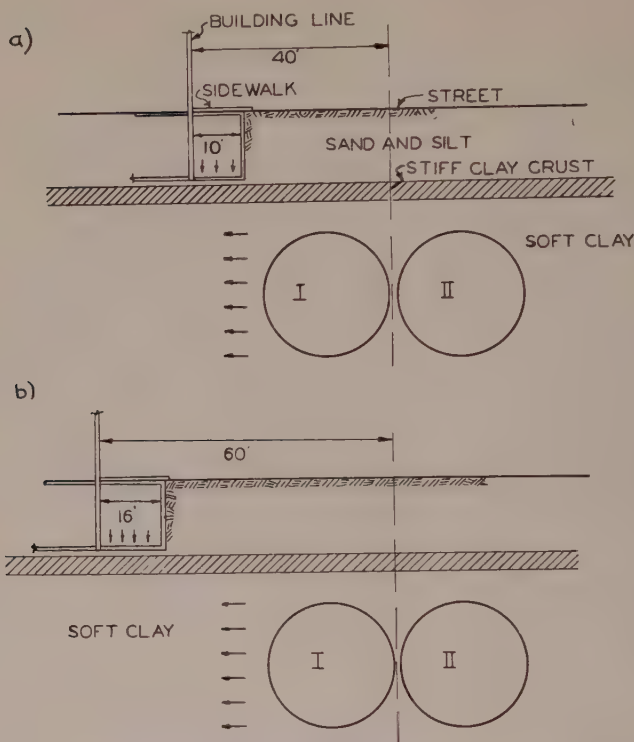


FIG. 15.—SIMPLIFIED SECTIONS THROUGH TUNNELS AND ADJOINING VAULTS

the floor slab combined are strong enough to resist this tendency. The stiff crust has a thickness of about 3 ft. and an average shearing resistance of about 1200 lb./sq. ft. Assuming that an important heave requires the formation of two vertical shear planes through the crust and the floor slab, one at the building line and one at the curb wall, the strength of the crust and of the floor slab has about the same restraining effect on the clay as a surcharge of about 800 lb. per sq. ft. for the narrowest and about 600 lb. per sq. ft. for the widest vaults. The pressure exerted by the advancing shield on the clay located in front of the shield was about 3600 lb./sq. ft. in excess of the pressure required to maintain in the clay a state without shearing stress (see section dealing with resistance against shoving). It has already been mentioned that the resistance of the Chicago clay against shear due

to forces of short duration is independent of the pressure on the surface of sliding and roughly equal to one-half of the non-confined compressive strength q_c of the clay. Hence the clay fails by shear as soon as the difference between horizontal and vertical pressure exceeds $\pm q_c$. The non-confined compressive strength of the clay located in front of the shield is about $q_c = 1200$ lb./sq. ft. The smallest lateral excess pressure compatible with a frontal excess pressure of 3600 lb./sq. ft. is $p_c = 3600 - q_c = 2400$ lb./sq. ft. This is the horizontal excess pressure which acts on the clay located beneath the floor of the vault. The clay fails and the floor of the vault heaves unless the difference between p_c and the pressure q on the surface of the clay is smaller than the compressive strength q_c of the clay. Hence the vertical confining pressure q must be at least $2400 - q_c = 1200$ lb./sq. ft. The resistance of the crust and the floor against shear failure represent the equivalent of a surcharge of 600 to 800 lb./sq. ft. as shown before. The balance of 600 to 400 lb./sq. ft. must be supplied by loading the floor. This was usually accomplished by loading the floor with a layer of sandbags, about 5 ft. thick. In most cases the sandbags reduced the heave to a few inches.

The estimate of the surcharge required to prevent the clay from rising into the vaults was based on an average value for the shearing resistance of the stiff crust of about 3600 lb./ft. of length of the potential vertical surface of shear. In some places the stiff crust was very much weaker, whereupon the floor of the vault rose in a few cases through a distance of several feet in spite of the sandbags. The observations in the vault of one building may serve as an example. The floor rose through a distance of almost 3 ft. in spite of the sandbags. After the heave occurred the bulge was sampled to a depth of 4 ft. below its surface. The tests showed that the water content of the clay within the stiff crust of the bulge ranged between 32 and 38%, whereas the average water content of the clay usually contained in the stiff crust is about 28%; which indicates that the stiff crust at the site of the heave was exceptionally weak.

A detailed study was made of the heave in the vaults of 37 buildings involving a total frontage of 6850 ft. The observations led to the following conclusions. The floor of sub-sidewalk vaults of buildings on spread footings heaved without sandbags between 6 and 12 in.

Where sandbags were used the depth of the layer of sandbags was made equal to about 5 ft. regardless of the width of the vault. Therefore in the narrow streets, where the vaults are narrow, the sandbags were usually effective, whereas in the wide streets, where the vaults are wide, the sandbags were not so effective. Fig. 16a illustrates a case

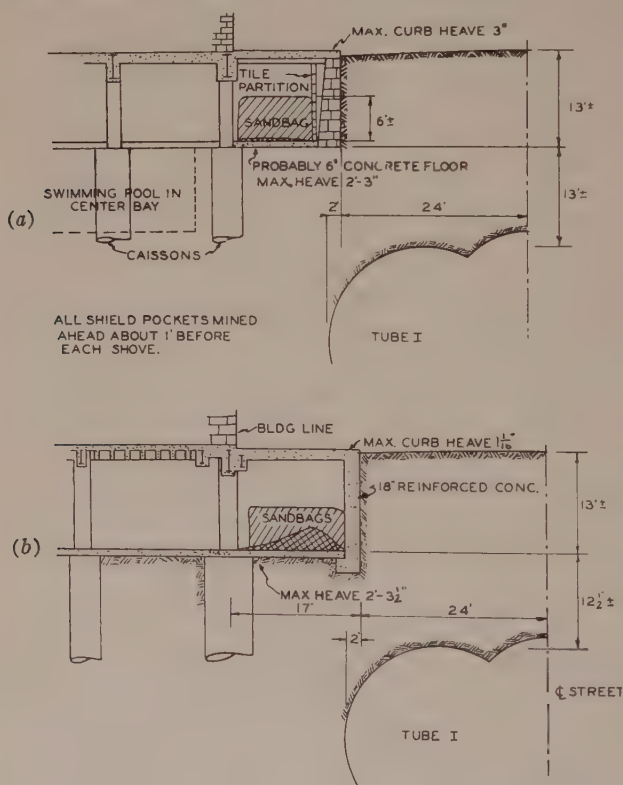


FIG. 16.—OBSERVED HEAVE OF CONCRETE FLOOR IN VAULTS

where the sandbags were effective (width of the vault 14 ft.). The presence of multiple story-sub-basements beneath the buildings or of a row of caissons beneath the building line usually increased the heave in the sub-sidewalk walls. A row of caissons beneath the curb wall had the opposite effect. This was to be expected. Walls and caissons represent relatively rigid obstacles which interfere with the deformation

tion of the clay and increase the pressure required to shove the shield. If these obstacles are located between the tunnel and the vault, as are the caissons beneath a curb wall, they take up part of the lateral pressure exerted by the clay in front of the shield and their pressure reduces the heave. On the other hand, if they are located beneath the main wall of the building, the presence of the obstacles increases the lateral pressure on the clay located beneath the vault and the clay fails unless the confining downward pressure at the floor of the vault is greater than the vertical force required to prevent a failure of the clay due to the increased lateral pressure.

These findings call one's attention to the forces which act on relatively rigid obstacles in the clay and to the danger of the failure of walls and caissons due to the pressure exerted by the advancing shield.

The horizontal pressure which acts on the 4-ft. reinforced concrete wall shown on the left-hand side of the tunnel in Fig. 17 can be esti-

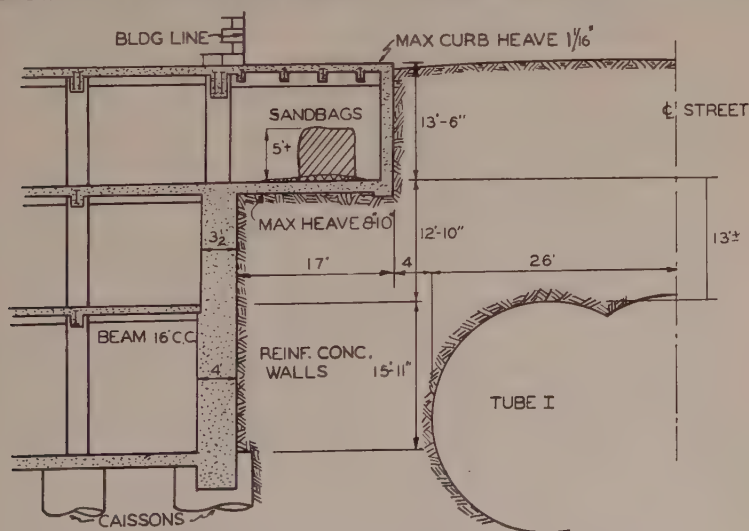


FIG. 17.—SHIELD SHOVING ALONG MULTI-STORIED SUBBASEMENT

mated in the following manner. Before the shield arrives in the proximity of the wall, the vertical unit pressure on a horizontal section at mid-height of the wall is approximately equal to 20 ft. $\times$ 110 lb./cu.

ft. = 2200 lb./sq. ft., wherein 20 ft. represents the vertical distance between the section and the floor of the vault. If there were no shearing stresses in the clay, the wall would be acted upon by a horizontal unit pressure of the same intensity. The advancing shield adds to this pressure another one, which increases the pressure in the clay to the point of failure as demonstrated by the rise of the floor of the vault. In order to rise into the vault the upward pressure of the clay must overcome the shearing resistance of the stiff crust and of the floor and the weight of the sandbags which have been introduced into the vault immediately before the shield approached the vault. All these resistances represent the equivalent of a vertical confining pressure of about 900 lb./sq. ft. of the floor of the vault. The non-confined compressive strength of the clay adjoining the wall is about $q_c = 1200$ lb./sq. ft. If the unit weight of the clay were equal to zero, the horizontal unit pressure required to produce a failure of the clay would be equal to the sum of q_c and the vertical pressure of 900 lb./sq. ft., or 2100 lb./sq. ft. The weight of the clay increases this pressure by 2200 lb./sq. ft. as shown before. Hence the total horizontal pressure on the wall is about 4300 lb./sq. ft. unless part of this pressure is transferred by arching onto the horizontal supports of the wall.

In one instance the shield passed within a distance of a few feet from a curb wall which had been underpinned prior to the passage of the shield. In order to obtain information on the pressure exerted by the advancing shield on this wall, a Carlson cell was installed on the street side of the curb wall at an elevation of about 6 ft. above the floor of the vault and at a horizontal distance of about 6 ft. from the center line of the adjoining needle beam. The shortest distance between the cell and the tunnel was 5 ft. Fig. 18 shows the effect of shoving the shield on the pressure indicated by the cell. The clay adjoining the cell was confined in the corner between a stiff vertical wall and a horizontal layer of stiff clay. The cell did not register any pressure until the cutting edge of the shield arrived within a distance of about 40 ft. from the cell. This observation indicates that the cell registered only the pressure in excess of the pressure which acted on the site of the cell prior to the installation of the cell. Nevertheless the pressure rose steadily as the shield approached the cell, up to a maximum value of about 5000 lb./sq. ft. After every shove of the

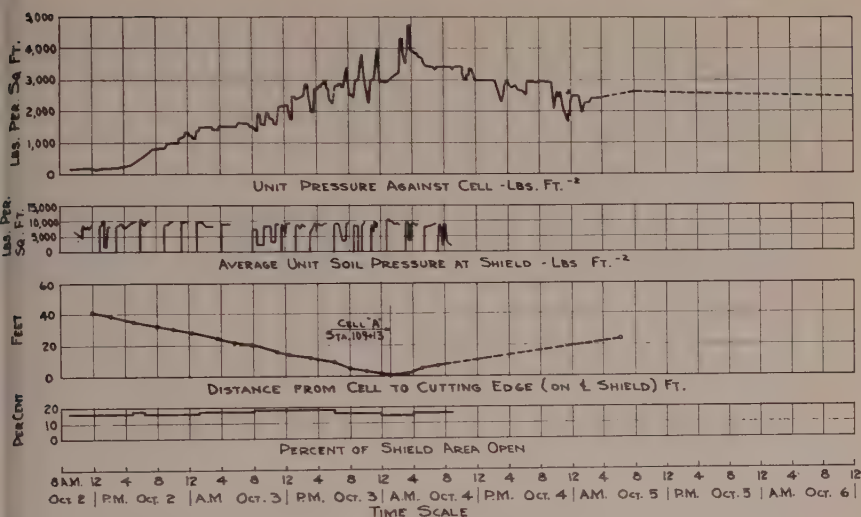


FIG. 18.—PRESSURE-CELL READINGS ON CURB WALL DURING PASSAGE OF SHIELD WITHIN 5 FT. OF WALL

shield the pressure on the clay in front of the shield dropped from 10,000 lb./sq. ft. to almost zero. Simultaneously the pressure registered by the cell decreased by about 1000 lb./sq. ft. as shown in the figure. The high value of the maximum pressure which acted on the cell illustrates the importance of the influence of confinement on the pressure in the clay. By using the method which has been employed for estimating the horizontal pressure on the reinforced concrete wall shown in Fig. 17, one finds that the horizontal excess pressure which acted on the clay beneath the vault during the passage of the shield was not greater than about 1200 lb./sq. ft. or less than one-fourth of what the cell indicated.

On account of the uncertainties associated with estimating the pressure on walls or caissons embedded in the clay next to the advancing shield, it is advisable to attempt, if possible, to measure the deflections of these obstacles during the approach of the shield. If the deflections become alarming or if an estimate indicates that the pressure may exceed the strength of the structure, the shield should not be shoved until part of the clay located in front of the shield has been removed by hand mining. Such a necessity arose on Contract

D-1, where the subway passes on a curve beneath the Selwyn and Harris Theaters, at the corner of Lake and Dearborn Streets. These two structures were founded on spread footings and were underpinned prior to subway construction by means of caissons to hardpan. A network of steel beams was placed below the level of the basement floor to support the various column and wall loads from the existing structure and to transfer them to the caissons. The bottom of these beams was approximately nine feet above the top of the shield. In several places the clearance between the shields and the caissons was not more than six inches. Mining in advance of the shield, particularly on the outside of the curve, was carried out consistently in order to negotiate the sharp curve and to prevent damage. With the exception of a few instances of heaves of two or three inches of the floor slabs, no disturbance caused by the shield mining was noted.

DESIGN OF THE TUNNELS

The design of the permanent lining of the tubes required certain assumptions regarding the intensity and the distribution of the pressure on the tubes. As a preliminary to a discussion of these assumptions, let us consider the pressure which acts on a closed tube with a square cross section, Fig. 19*a*, which is entirely surrounded by a heavy liquid with a unit weight γ . The tube is filled with air under atmospheric pressure and the weight of the tube is assumed to be negligible compared to the weight of the liquid displaced by the tube. The shaded areas on the left-hand side of Fig. 19*a* represent the calculated fluid pressure on the walls of the tube. The pressure on the roof of the tube is $2 P_r = 2 a \gamma h$ and that on the bottom of the tube is $2 P_b = 2 a \gamma (h + 2a)$. The difference between these two pressures is

$$2(P_b - P_r) = 4 a^2 \gamma \quad (1)$$

which is equal to the weight of the liquid displaced by the tube. Hence the tube would float to the surface unless it is held down by a force equal to the weight of the liquid displaced. This force is equal to

$$2 P = 4 a^2 \gamma$$

If the tube is surrounded by a mass which has some shearing strength such as soft clay the force $2P$ which is necessary to hold the tube in place is supplied by shearing stresses in the clay. The force S_1 on the right-hand side of Fig. 19*a* represents the total shearing force on

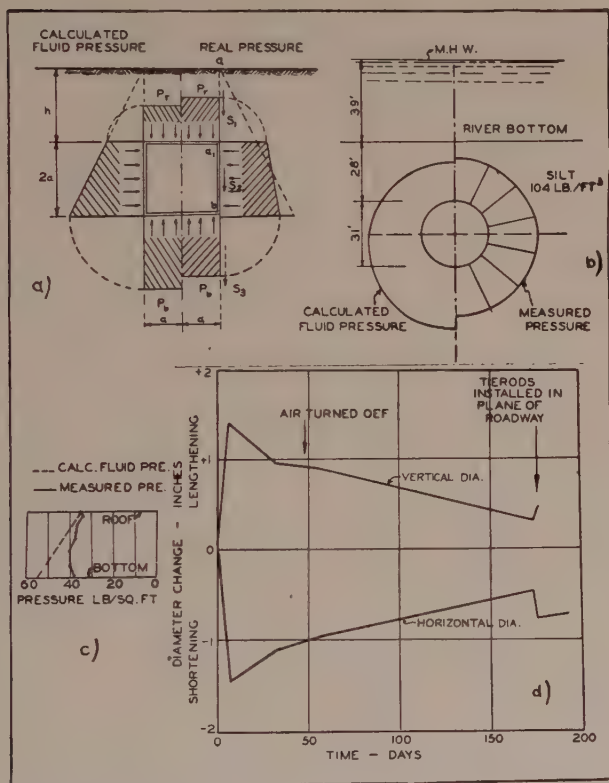


FIG. 19.—(a) TO (c) EXTERNAL PRESSURE ON TUNNEL TUBES IN VERY SOFT CLAY
 FIG. 19.—(d) CHANGE OF DIAMETERS OF LINCOLN TUNNEL PLOTTED AGAINST TIME

aa_1 and S_2 is the shearing force which acts on the side of the tube. The sum of the forces, $S_1 + S_2$, must be equal to one-half of the hydrostatic uplift $2P = 4a^2\gamma$, whence

$$S_1 + S_2 = P = 2a^2\gamma$$

Assuming, as a crude approximation, that the shearing stresses, s per unit of area, are uniformly distributed, one obtains

$$S_1 + S_2 = 2a^2\gamma = (2a + h)s$$

or

$$s = \frac{2a^2\gamma}{2a + h}$$

On account of the existence of the forces $S_1 + S_2$ the pressure

on the roof of the tunnel is $2 P'_r$ instead of $2 P_r$ and the pressure on the bottom is $2 P'_b$ instead of $2 P_b$. Equilibrium of the soil located above the tube requires

$$P'_r = a \gamma h + S_1$$

which is greater than P_r and the equilibrium of the tube requires

$$P'_b = P'_r + S_2$$

The difference

$$P'_b - P'_r = S_2 = 2a^1 \gamma \frac{2a^2}{2a + h} = 2a^2 \gamma \frac{2a}{2a + h}$$

is smaller than the difference $P_b - P_r = 2a^2 \gamma$ (eq. 1). These computations show that the shearing forces which prevent the rise of the tube increase the pressure on the roof of the tube and they reduce the pressure gradient on the sides as shown on the right-hand side of Fig. 19a. Owing to the influence of the shearing forces, the pressure on the tube roughly resembles an external pressure which acts in every direction with equal intensity.

In the preceding estimate illustrated by the right-hand side of Fig. 19a, it was assumed that the shearing stresses on the vertical section ac are equal to zero below the level of the bottom of the tunnel at point b . In reality this part of the section is also acted upon by a shearing force S_3 , indicated by a dashed arrow. The force S_3 further reduces the pressure on the bottom of the tunnel. If the clay is fairly stiff, this pressure becomes equal to zero. This condition exists during the construction stage in all the liner-plate tunnels. Otherwise the bottom of these tunnels would rise.

Conditions similar to those illustrated by the right-hand side of Fig. 19a also prevail if the tube is cylindrical. This conclusion is in accordance with the results of pressure measurements on the Lincoln Tunnel.³ In Fig. 19b the normal pressure on the wall has been plotted from the walls of the tunnel as a base line in a radial outward direction. The left-hand side shows the intensity and distribution of the calculated fluid pressure and the right-hand side the measured pressure (July 8, 1935, air pressure 16 lbs./sq. in., silt cover over the tunnel 28 ft.). In Fig. 19c the ordinates represent the depth below the roof of the tunnel, the abscissas of the dashed line the calculated fluid

pressure at different depths, and those of the plain line the corresponding measured normal pressure (August 29, 1935, air pressure zero, silt cover 19 ft.). In accordance with the conclusions derived from Fig. 19*a*, the increase of the measured pressures with depth was very much smaller than that of the computed fluid pressures. For this reason the bending moments produced by the real pressure in a cylindrical tube should be unimportant. The bending moments produced by a perfectly uniform all-around pressure in a cylindrical tube are equal to zero. Fig. 19*d* shows the change in the length of the diameters of the Lincoln Tunnel. The outside diameter of the tubes was 31 ft. and the thickness of the shell was only 1 ft. 6 in., which indicates that the walls were relatively flexible. Nevertheless the permanent change of the diameter of the tubes did not exceed 0.7 in. or 0.2 per cent of the length of the diameter.

Figs. 19 *b* to *d* represent practically everything that was known in 1938 regarding the pressure on shield tunnels. The Lincoln Tunnel is located in a bed of almost fluid and fairly homogeneous silt. It was probable, but by no means certain, that the distribution of the pressure on shield tunnels driven into a mass of clay with a variable consistency, located beneath a stiff crust, would be similar to that shown in Fig. 19*c*. Therefore it was necessary to design the shield tunnels on the basis of the empirical rules which have previously been developed in connection with the sewer tunnels of Chicago, although those tunnels have been constructed by means of the liner-plate method.

In accordance with these rules, the vertical pressure above the tunnel was taken as the weight of the overburden, on the base of the tunnel as the weight of the overburden plus the weight of the arch, and on the sides as the pressure exerted by a fluid having a density which varied from $1/3$ to $2/3$ that of the actual soil. The result was a reinforced concrete tube 2'-2" in thickness. Yet, other empirical rules led to a thickness of only 1'-6" for the Lincoln Tunnel, although its diameter exceeds that of the subway tubes by 6 feet, and it is doubtless entirely safe. In order to obtain definite information concerning the real conditions, and to secure the data required for a less costly design on future contracts, it was decided to establish an experimental shield section on Contract S-3. An abstract of the results of the observations and of their bearing on design is contained in the following section.

OBSERVATIONS ON THE EXPERIMENTAL SECTION AND THEIR BEARING ON THE DESIGN OF SHIELD TUNNELS

The experimental section is located in the West tube on Contract S-3 between Sta. $87 + 55$ and $88 + 00$. The section consists of the segmented steel lining which was ordinarily used as a temporary lining. The first ring was erected on April 20, 1940. The second (East) shield passed the section on June 16, 1940. The air was removed on July 11, 1940. In order to maintain the lining in its original, relatively flexible state, no concrete invert was poured. Immediately after erection and at suitable time intervals after erection of the steel lining, the following observations have been made: (1) Measurement of the horizontal and the vertical diameter of the lining; (2) Strain gage measurements on different parts of the lining; (3) Determination of the shape of the cross-section of the lining; and (4) Measurement of the pressure in the temporary drums which have been introduced into the experimental section during the passage of the second shield. Seven months after the air was turned off, water pressure gages were installed and the pore water pressure in the clay was measured at different points at different distances from the lining, as shown in Fig. 14.

The measurements of the horizontal diameter were made on 18 rings, starting at the time when the rings were installed within the tail piece of the shield. In this connection it should be mentioned that the rings were erected out-of-round in such a manner that the horizontal diameter was about 1 in. shorter than the vertical one. This type of deformation was conditioned by the fact that the cylindrical tail piece of the shield was somewhat out of shape. In order to prevent the rings from assuming their natural, circular shape during erection, the clearance between the segments and the tail piece of the shield was filled out with hemp ropes.

The vertical diameter was measured on six rings in such a manner that the displacement of the tube could also be measured. In general, the lengthening of the horizontal diameter was equal to the simultaneous shortening of the vertical one and vice versa, which indicated the absence of an appreciable change of the length of the circumference of the rings. The error in measuring the diameters did not exceed $1/16$ in.

In Fig. 20 is shown the relation between the time and the change

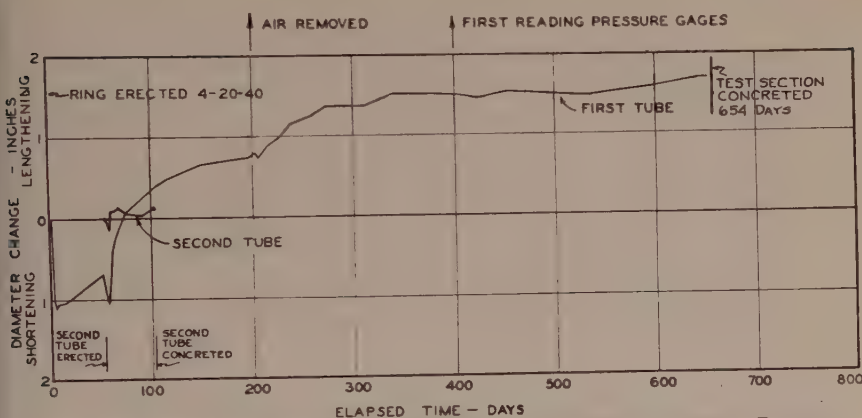


FIG. 20.—CHANGE OF HORIZONTAL DIAMETER OF EXPERIMENTAL SECTION PLOTTED AGAINST TIME

in length of the horizontal diameter of the ring 30 *N*. Within the first few days after the rings were assembled, the diameter of the rings steadily decreased. The greatest shortening ranged between 1 and $1\frac{1}{2}$ in. This process corresponds to the initial shortening of the horizontal diameter of the Lincoln Tunnel, Fig. 19*d*. After the initial period of shortening the horizontal diameter of the tube increased and after about one year assumed a constant value, which was about $1\frac{1}{2}$ in. in excess of the initial length of the diameter. It has been mentioned before that the initial horizontal diameter was shorter than the diameter of a true circle whose circumference is equal to that of the ring. Hence the final shape of the rings was almost circular. The curve which represents the relation between time and diameter change shows two breaks. One of them corresponds to the time $t_1 = 60$ days, at which the second shield passed the experimental section, and the second one corresponds to the time $t_2 = 200$ days at which compressed air was turned off. Each one of these two events produced a temporary but very conspicuous increase of the rate at which the ring changed its shape.

A circular ring does not permanently retain its circular shape under external pressure unless one of the following two conditions is satisfied. Either the ring is so stiff that it remains circular in spite of a non-uniformly distributed all-around pressure, or else the ring is

acted upon by a uniformly distributed all-around pressure. In order to decide which one of these two conditions was satisfied in reality, one must examine the flexural rigidity of the ring. Information regarding this rigidity can be obtained from the results of the strain gage observations.

Each ring consisted of six segments with bolted connections. Fig. 6a shows a side view of one of the segments and Fig. 6b a cross section. Each one of the segments of each one of two rings in the experimental section was equipped with eight sets of gage lines for strain measurements as shown in Fig. 6b which were so located that the readings gave a fairly complete picture of the state of stress in the ring. One of the cross sections through one of the rings was equipped with 15 gage lines which made it possible to determine the distribution of the stresses over the section. The observations on this particular section showed that the inner flanges carried very little stress which indicated that the steel contained in these flanges was hardly utilized. Furthermore, there is no evidence that the steel skin which represented the lining proper participated to a considerable extent in carrying the bending stresses. In general the distribution of the stresses over radial sections through the segments has no resemblance to what one should expect on the conventional assumption that the bending stresses increase in direct proportion to the distance from the neutral axis. Considering these facts, it is the writer's opinion that the effective moment of inertia I of the rings was hardly greater than about 70 in.⁴, which is one-third of the theoretical value calculated on the assumption that all the parts of the segments participate fully in carrying the bending stresses. Retaining the figure $I = 70$ in.⁴ and disregarding the weakening of the rings by the bolted connections, one finds that a difference of less than about 400 lb./sq. ft. between the horizontal and the vertical unit pressure on the outside of the tube is sufficient to shorten the horizontal diameter by one inch. In reality, the rings consisted of six segments with bolted connections. These connections further reduced the flexural rigidity of the rings. The weakening effect of the joints is demonstrated by Fig. 21. Fig. 21a shows the deviation of the shape of the ring from a true circle approximately at the time of the greatest shortening of the horizontal diameter, 47 days after erection. In Fig. 21b the radial displacements of the reference points which

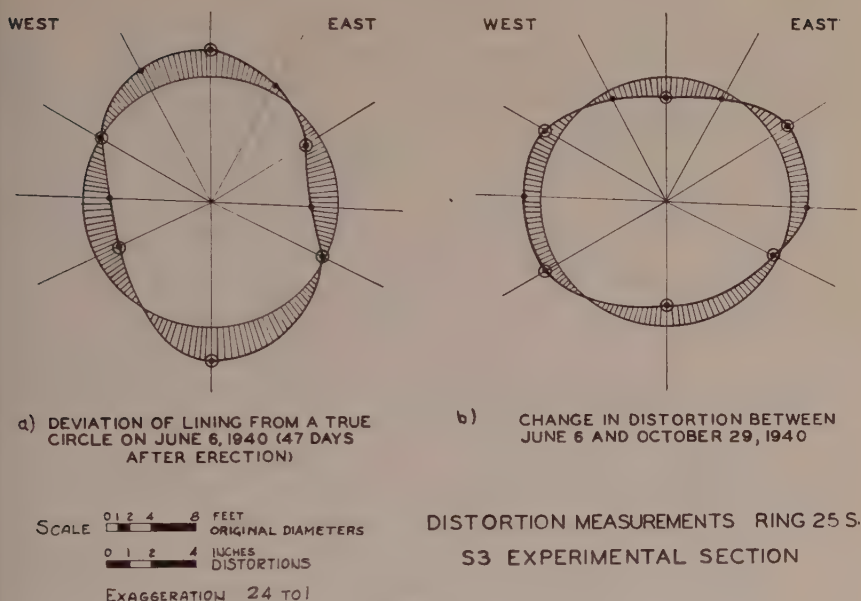


FIG. 21

occurred during the subsequent 145 days have been plotted in radial directions from a circular reference line. Fig. 21b shows that the joints are very much weaker than the segments. Otherwise the curve which represents the differential deformations would be smooth and elliptical. The weakness of the joints is to be expected because the bolted connection between the segments, shown in Fig. 6 is far from representing a full structural equivalent for a jointless connection. Hence the pressure difference required to change the length of the diameters by one inch was undoubtedly very much smaller than 400 lb./sq. ft.

The facts mentioned in the preceding paragraph indicate that the flexural rigidity of the rings as a whole was rather insignificant. If we disregard the flexural rigidity entirely, the progressive change of the diameter illustrated by Fig. 20 can be interpreted in the following manner. During the first days after the ring emerged from the shield, the clay advanced from both sides towards the tunnel and shortened the horizontal diameter by about $1\frac{1}{2}$ in. Nevertheless, the

shape of the ring was almost circular, because the difference between the length of the two principal diameters was smaller than 2%. A perfectly flexible tube with that shape is not in a state of equilibrium, unless the pressure in the clay adjoining the tube has practically the same value p_0 at every point of the surface of contact between the clay and the tube. If the ring has some flexural rigidity, the pressure which acts in the direction of the shorter axis must be somewhat greater than the pressure which acts at a right angle to this direction.

The subsequent elongation of the horizontal diameter indicates that the state of stress described above caused a gradual consolidation of the layer of remolded clay located on both sides of the tube. The rate of consolidation depends on the conditions of drainage, everything else being equal. Hence, if the conditions of drainage change, the rate of the increase of the length of the horizontal diameter should also change. This conclusion is confirmed by observation. (Fig. 20.) Before the passage of the second shield, the remolded clay could drain only into the layer of pea gravel surrounding the first tube. During the passage of the second shield a layer of pea gravel was placed on the east side of the remolded layer, permitting a drainage of this layer in two directions. Therefore the passage of the second shield increased very considerably the rate of lengthening of the horizontal diameter of the first tube. A second temporary increase of the horizontal diameter occurred when the air was turned off, because this event reduced the pore water pressure at the surface of contact between the remolded clay and the pea gravel from about 1800 lb./sq. ft. to zero. At the time when consolidation was complete, the shape of the rings was almost perfectly circular. Hence the bending stresses in the tube were almost equal to zero. This conclusion was confirmed independently by the results of the strain gage measurements.

During the process of deformation illustrated by Fig. 20 the clay was supported only by the steel lining. When a shield tunnel is constructed for transportation purposes, a permanent reinforced concrete lining is added at some time after the erection of the rings. The construction of the concrete lining increases very considerably the flexural rigidity of the tube. The bending stresses which develop in the concrete shell depend on the time when the shell is constructed and on the thickness of the shell. Since the final bending stresses in a per-

fectly flexible tube are equal to zero, the bending stresses in the concrete shell increase with increasing thickness of the shell, everything else being equal. If the shell is thin enough to withstand a total increase of its diameter by about two inches without cracking, no reinforcement is required in excess of what is needed to take care of the shrinkage and temperature stresses. Therefore both economic and structural considerations lead to the rule that the shell should be made as thin as compatible with requirements of construction. While theoretically a shell with a thickness of less than one foot would be ample, practical considerations require greater thickness. The shell of the Lincoln Tunnel with a diameter of 31 ft. was 1 ft. 6 in., whereas that of the Chicago tunnels with a diameter of 25 ft. was 2 ft. 2 in. Engineering considerations suggest in addition that the shell should be constructed as late as possible. According to the writer's judgment, the bending moments are insignificant even in a fairly thick shell and a nominal reinforcement should be sufficient to take care of them. Deformation of the tube automatically reduces the bending moments.

The time which elapsed between the erection of the rings in the second tube and the construction of the reinforced concrete lining in this tube was about 50 days. The relation between time and the change of the horizontal diameter for the second tube is also shown in Fig. 20. It had no resemblance to that for the first tube. During the first few days the length of the diameter increased and then it decreased. Immediately prior to the concreting of the shell the difference between the initial and the final diameter was nowhere greater than about $1/6$ in. Therefore the stress conditions in the lining of the second tube seem to be considerably more favorable than those which develop in the first tube.

The shield in Chicago was shoved through a mass of non-homogeneous clay located beneath a stiff crust, whereas the shield on the Lincoln Tunnel (Ref. 3) was shoved through a fairly homogeneous mass of very soft silt. Nevertheless the relation between time and the change of the diameter and the distribution of the external pressure over the outside of the tubes was remarkably similar. Therefore the preceding conclusions concerning the design of shield tunnels seem to be applicable to very different conditions.

CONCLUSIONS

(1) The heave produced by shoving the shield in Chicago was decisively influenced by the presence of a stiff crust of clay located about halfway between the roof of the tunnels and the surface of the street. Therefore the effect of shoving on the street and on adjoining buildings was different from what one should expect when shoving a shield through a homogeneous mass of soft clay.

(2) The size of the openings in the shield varied between 4% and 20% of the area covered by the shield and the rate of shoving varied between 30 and 300 minutes per shove or 12 and 120 minutes per foot. On account of the stiff crust mentioned above, neither the percentage opening nor the rate of shoving had a marked influence on the heave. Therefore it was necessary to resort to hand mining as often as the observations on the street surface indicated a tendency towards increasing heave. On account of the care with which these observations were made, it was possible to keep the heave within remarkably narrow limits regardless of the external conditions. The percentage heave ranged between 0.6 and 2.4% of the heave which would occur when shoving with a closed shield. The corresponding maximum rise of the street surface ranged between 1 in. and 4 in. Whenever there was a tendency of the heave to become greater than 4 in., the heave was reduced by hand mining in front of the shield.

(3) The heave at a given station due to the passage of the first shield was always followed by a settlement which was greater than the preceding heave. The settlement was partly due to the stretching of the clay adjoining the boundary between the lining and the tail end of the advancing shield and partly due to the incipient consolidation of the remolded layer of clay which surrounded the lining.

(4) The heave at a given station due to the passage of the second shield was followed by a progressive settlement of the street surface which continued at a decreasing rate until about one year after the passage of the shield. It was exclusively due to the gradual consolidation of the remolded layer of clay which surrounds the tubes. It amounted to several inches. The thickness of the remolded layer is considerably smaller than 4 ft. Beyond the outer boundary of the remolded layer the strain produced in the clay by shoving the shield is insignificant. One cannot shove a shield without remolding the clay

located in the vicinity of the tube. Therefore the settlement referred to in this and in the preceding paragraph must be considered inevitable.

(5) Shoving the shield caused a heave of the floor of the sub-sidewalk vaults of adjoining buildings. The heave was resisted by the shearing strength of the floor and of the stiff crust. By temporarily covering the floor of the vaults with a five-foot layer of sandbags it was possible to reduce the heave to a few inches unless the vault was exceptionally wide or the stiff crust of clay exceptionally weak.

(6) The pressure transmitted by the advancing shield through the clay to semi-rigid bodies such as caissons or walls embedded in the clay depends on the degree of confinement of the clay adjoining the obstacles. Therefore the position of such a body with reference to the cutting edge of the shield is only one of several equally important factors which influence the intensity of the pressure. On account of the uncertainties involved in estimating the pressure, it is advisable to observe the deflections of the body during the passage of the shield. If it is doubtful whether the body is able to stand the pressure without breaking it is necessary to mine by hand in front of the shield, until the shield is advanced to a distance of about 20 feet beyond the station at which the body is located.

(7) The shape of the tunnel tubes changes slightly until about one year after the erection of the rings. After conditions become stationary, the tunnel tubes are acted upon by a fairly uniformly distributed external pressure. Hence, at that stage, the bending moments in a cylindrical tube with a circular cross section are very small. Both economic and structural considerations lead to the rule that the permanent lining should be made as thin as compatible with construction requirements.

ACKNOWLEDGMENTS

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THE SMITH-PUTNAM WIND TURBINE PROJECT

BY JOHN B. WILBUR, MEMBER*

(Presented at a meeting of the Boston Society of Civil Engineers held on March 18, 1942.)

GENERAL

THE Smith-Putnam Wind Turbine Project is a large scale experiment in which it is hoped to find out whether or not the energy of the winds can be harnessed and converted into electrical energy economically and on a large scale. The project was conceived by Palmer Cosslett Putnam, of Boston, and is being sponsored by the S. Morgan Smith Co. of York, Pennsylvania. It is from these two names that the title of the project is derived.

The use of wind power is not new. The use of wind to propel sailing vessels represents one of the early applications of the forces of nature by man. Later, windmills were built to supply power for pumping water and for other purposes. Quite recently a large number of small farm windmill units have been built, developing D.C. current which is stored in storage batteries for periods of no wind. These farm units are used in isolated locations where no other electrical energy is available. From the economic viewpoint they do not compete with low priced electrical energy as developed in large steam and hydro-electric installations.

There are perhaps two main reasons why the test unit of the Smith-Putnam Wind Turbine, which has been built and is being tested on Grandpa's Knob, near Rutland, Vermont, has attracted a certain amount of interest.

Undoubtedly the fact that this wind turbine is by far the largest windmill type of structure ever built, is an important factor. Mere size, in itself, is not or should not be of too much interest to engineers, but when economic considerations show that great size is consistent with good economics, interest in a structure is increased accordingly. The two blades of the test unit have a diameter of 175 feet and are

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mounted on a tower in such a manner that the hub of the turbine is approximately 120 feet above the ground. The tip of the blade, as it passes over the tower, is about 210 feet above the ground, which is as high as a typical 18-story office building.

A second reason, and a more important one from the engineering viewpoint, for the interest in this project, results from the fact that in so far as is known, alternating current is being developed from wind energy for the first time. This requires careful speed regulation, since it becomes necessary to drive the A.C. generator, which in this instance is synchronous, at a constant speed, while wind conditions are varying, in order that the frequency of the system to which the wind turbine is connected may be matched. Some idea of the difficulty of this control problem may be realized from the fact that wind energy varies with the cube of wind velocity. Since wind velocity may change by as much as 50% in one second, the effective head on the turbine may vary by $(1.5)^3$ or over 300% in one second. An important factor in making speed control possible in the Smith-Putnam Wind Turbine is the large moment of inertia about the blades, resulting in a unit inertia far greater than encountered in hydraulic turbines. In the test unit, speed control is achieved by means of a speed governor driven from the main turbine shaft. This governor actuates valves in a 400 lb. pressure hydraulic system which operates a servo motor located in the tail piece of the unit. The piston of the servo motor is connected by links to torque tubes, by means of which the pitch of the turbine blades is changed. The torque delivered by the blades varies from zero, when the blades are in the feathered position—that is, when the chords of the blades are approximately normal to the plane of rotation, to a maximum when the blades are in the design blade angle position—that is, when the chords of the blades make an angle of approximately 3° with the plane of rotation. The linkage from the servo motor to the blade torque tubes is such that at any instant both blades have the same blade angle.

One of the questions most commonly asked in connection with this project is—"What do you do when there isn't any wind?" The answer to this is simple. The wind turbine develops no power. This particular wind turbine is designed to develop 200 kilowatts in a 20 mile wind and to reach its rated capacity of 1000 kilowatts in a 30 mile

wind. Thus a wind turbine, by itself, can develop only secondary energy, although the amount of secondary energy which can be developed in a given period of time is predictable within certain limits on the basis of synoptic meteorology. If, however, wind turbines are interconnected with hydroelectric stations, certain hydroelectric units can be shut down when the wind is blowing and electrical energy can be obtained from the wind. This results in the saving of stored water, which can be used during periods of no wind. In this manner the primary power output of the total system—water plus wind, combined—can be raised. In effect, when the wind is blowing, wind energy can be stored by means of the water withheld in the reservoirs.

DESCRIPTION OF TEST UNIT

The general layout of the test unit is shown in Fig. 1. It will be seen that both the turbine proper and the power house are mounted on a tower. This tower is very substantial in its construction, each of the four legs being 14-inch, 246 lb. Carnegie beams. The total height of the tower is 110 feet. The top section of the tower consists of a welded tower cap, which houses large anti-friction bearings, which support a vertical pintle shaft about which the entire mechanism aloft rotates as the wind changes direction.

It will be noted that the turbine blades are on the downwind side of the tower. This is believed to be more efficient aerodynamically, and it also makes possible coning action of the blades. At the root of each blade assembly there are coning hinges, about which the blade may turn during turbine operation. At any instant the coning angle of the blade is determined by the effect of the aerodynamic, centrifugal and gravity forces acting on the blades. The two blades cone independently. This coning action is damped by means of coning damping cylinders which are connected to the blades through coning linkages. The test unit is designed so that there is freedom for blade coning of 20° measured downwind from the plane normal to the axis of rotation and 4° upwind, measured from this same plane. The main axis of the turbine makes an angle of $12\frac{1}{2}^\circ$ with the horizontal, so that gravity brings the blades to rest against the upwind coning stops when the unit is not rotating.

Because the blades are on the downwind side of the tower, the unit



FIG. 1.—SCHEMATIC DRAWING, SHOWING ARRANGEMENT OF TEST UNIT

does not follow the wind direction—that is, it does not yaw, from its own aerodynamic stability. Mounted on the upper upwind end of the structure aloft, a damped yaw vane follows general wind direction and in so doing actuates valves in a hydraulic yaw system which drives a hydraulic motor which is geared to a large bull gear which is fastened

to the top of the tower cap. This yaw mechanism forces the unit to rotate and follow the general wind direction.

The main turbine shaft is supported on two large anti-friction bearings. The rated speed of the turbine blades is 28.7 r.p.m. At this speed of rotation, the tip speed of the blades is about 200 m.p.h. and the effective gravity at the tip of the blade about 30 g. The speed of the main shaft is stepped up through a gear box, marked *A* in Fig. 1, to approximately 600 r.p.m. The high speed shaft from the gear box is connected to the generator through a hydraulic coupling. This coupling has a slip-torque characteristic which varies almost linearly and is approximately 5% at rated turbine output. The slip in this hydraulic coupling permits different turbine speeds for different power outputs of the generator, thus making the action of a speed governor possible. The generator has a rated capacity of 1000 kilowatts and is driven at 600 r.p.m.

The bearings of the main turbine shaft, the main gear box, the generator and the other mechanisms aloft are supported on a pintle girder structure which is rigidly connected to the pintle shaft.

The structural members over the power house, supporting a service crane, were removed after the erection of the unit was completed.

SITE SELECTION

While many problems were encountered in the design of the test unit, there was in general a certain amount of precedent from other structures and mechanisms which helped greatly in arriving at various design decisions. In the important matter of site selection, however, it was necessary to initiate a comprehensive investigation of a new field—namely, the relation between topography and wind energy. Because of the importance of site selection, the investigation was approached by a number of methods. Long term wind power studies were conducted on the basis of weather maps under the direction of Dr. Sverre Petterssen, of M. I. T. The Central Vermont Public Service Corporation had shown interest in the project and had agreed to make available the facilities of their lines for the test. It was felt essential that the test should be carried out with a wind turbine tied into an actual power system so that the value of the power produced to that system could be determined. While Dr. Petterssen's studies covered

most of the western hemisphere, particular attention was paid to the wind regimes in the Vermont area. In order to correlate Dr. Peterssen's long term studies with actual wind conditions in Vermont, field measurements of wind velocities were carried out at thirteen different sites in the Green Mountains, under the direction of Dr. Karl O. Lange, who at that time was a member of the staff at Harvard. Anemometer masts, ranging in height from 75 to 110 feet, were constructed at each of these thirteen stations, and wind records obtained with Stewart cup anemometers. Balloons were released from a number of points and their courses charted, in order to obtain information concerning the flow of air currents over mountainous country. An independent approach to the matter of site selection was based on a correlation of tree growth with the magnitude and direction of prevailing and maximum winds.

At a meeting, held in Akron in June of 1940, all these data were considered and Grandpa's Knob chosen as the site for the test. This site was not chosen because it was believed to have a higher power output than other sites which were being considered, but because it combined a number of virtues. It was believed that the power output would be at least representative of average sites of similar elevation for the area under consideration, and its relatively low elevation, 2000 feet, made it possible to design the test unit for lower maximum ice load and wind velocity than would have been necessary at a higher site. The top of Grandpa's Knob was relatively open and required little timbering in order to give the wind a clean sweep. It was relatively accessible, it being necessary to build only 1.8 miles of road to reach the top.

Grandpa's Knob is a knoll on Biddie Ridge, which extends north and south about eight miles west of Rutland, Vermont. To the west there is a clear sweep across Lake Bomoseen and the Champlain Valley to the Adirondacks. To the east, there is a wide valley, beyond which are higher mountains of the Green Mountain Range, including Pico Peak.

DESIGN AND FABRICATION

The design and fabrication of the test unit was carried out by the S. Morgan Smith Company, with the cooperation of a number of well

known companies and with the help of many consultants. Dr. Theodore von Karman, of the California Institute of Technology, served as aerodynamic consultant. He was assisted by Mr. John F. Haines, of Engineering Projects, Inc., of Dayton, Ohio. Under Dr. von Karman's supervision, wind tunnel tests were carried out at Stanford University, which led to the selection of the blade form adopted. The blades were designed and fabricated by the Edward G. Budd Manufacturing Co., of Philadelphia, with Professor Joseph S. Newell, of M.I.T. as consultant. The pintle girder and a large portion of the mechanism mounted on it were designed and fabricated by the Wellman Engineering Company, of Cleveland, Ohio. The electrical designs, which included new developments in connection with the automatic switchgear, were carried out by the General Electric Company. The General Electric Company furnished all the electrical equipment, including the switchgear and generator. The tower was designed and fabricated by the American Bridge Company, of Pittsburgh, Pennsylvania. Heavy field erection was also carried out by this company. The governor and some of the auxiliary equipment in connection with speed control were designed and built by the Woodward Governor Company, of Rockford, Illinois.

Since a number of factors entered into the design, concerning which definite data were not available, an effort was made to over-design the test unit, both structurally and mechanically. The structural design was based on a maximum wind of 140 miles per hour without ice and on a wind of 100 miles per hour with an ice coating $4\frac{1}{2}$ inches thick over the entire structure, except for the blades, where a special ice loading was assumed, tapering from maximum thickness at the leading edge of the blade. Mechanically, an attempt was made to over-design the unit by the introduction of a large number of safety devices. As an example of these safety devices, mention might be made of the precautions which have been taken to prevent the turbine from reaching a runaway speed due to the inability of the hydraulic system to change the pitch of the blades in event of a failure of some kind. The main oil pump, which pumps the oil to the accumulator tank, is driven from the main shaft of the turbine. In event that this pump should fail to operate properly, or if its capacity should prove to be insufficient at any time, there is an auxiliary oil pump, driven from

the power of the high line. In event both pumps should fail, or if for any other reason oil pressure is lost, there is an inherent safety in the design of the pitching mechanism. The aerodynamic torque of the wind pressure acting on the blades is greater than the centrifugal torque which tries to hold the chord of the blades in the plane of rotation. It requires oil pressure to keep the blades in the design blade angle position. If this oil pressure is lost, the aerodynamic forces will partially feather the blades and slow the unit down.

Fig. 2 shows a blade being assembled in a jig at the Budd Manufacturing Company in Philadelphia. It will be noted that the main load carrying element of the blade consists of a longitudinal spar which is rectangular in cross section. The spar, itself, was fabricated by the

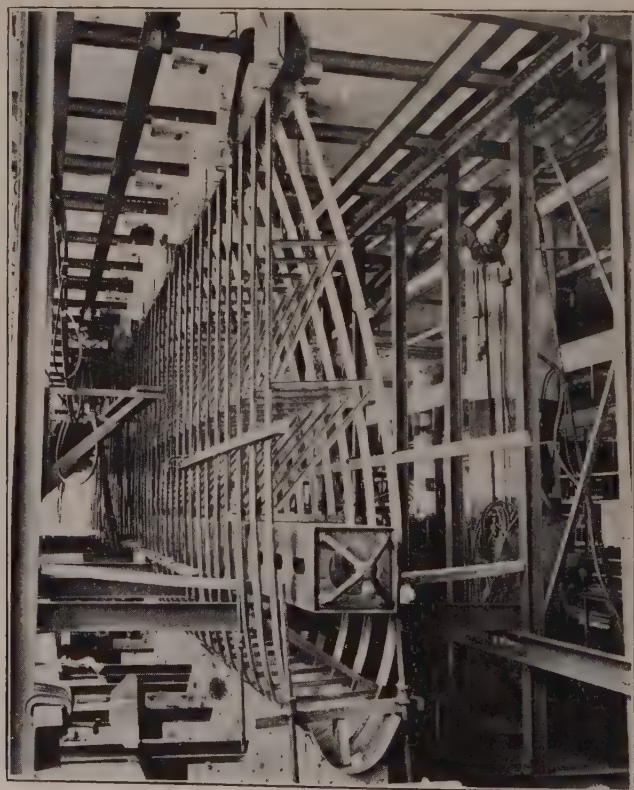


FIG. 2.—BLADE FABRICATION AT BUDD MFG. CO.

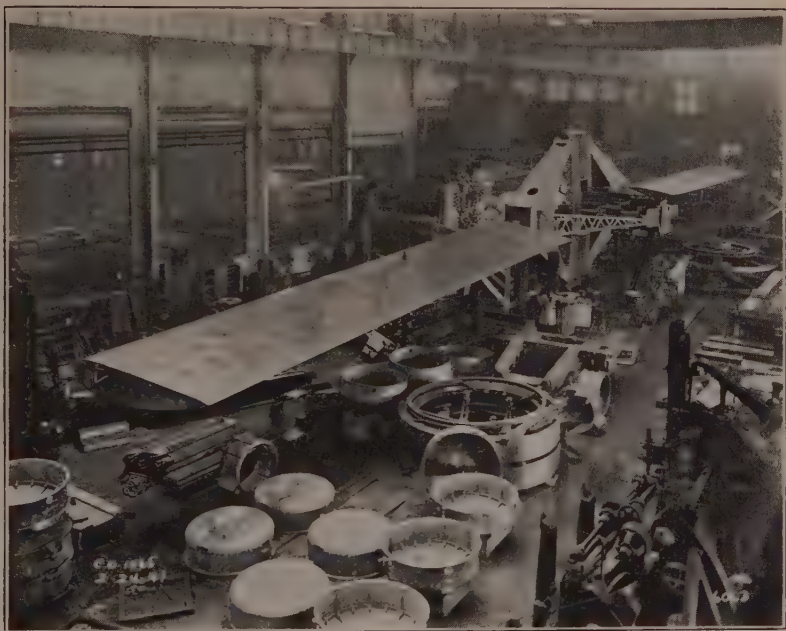


FIG. 3.—SHOP ASSEMBLY AT THE WELLMAN ENGINEERING CO.

American Bridge Company and is of welded construction, being built of a copper nickel alloy. Transverse to the spar are light stainless steel truss ribs. The skin of the blade consists of thin sheets of stainless steel, varying in thickness from .030 in. to .070 in.

After the blades were fabricated, they were shipped to Cleveland, where an entire shop assembly of the pintle girder and the mechanisms mounted thereon was made. This shop assembly was considered extremely desirable since it reduced field erection difficulties.

HAULING AND ERECTION

The material for the wind turbine was delivered at the West Rutland Branch of the Vermont Marble Company, where suitable cranes were available for unloading. This was important, since a number of the units shipped weighed from thirty to forty-five tons. The loads were hauled over the State highway to Castleton and thence over a County road to the base of Grandpa's Knob. On this County road there were five bridges of very light construction, all of which had to be greatly strengthened. This strengthening was accomplished by

means of timber bents and by laying additional planks to strengthen the floors. The road up Grandpa's Knob had been laid out with the intention of limiting maximum grades to 12%, but in a number of places there were actually grades of 15%. For the heavy loads it was necessary to have a tractor pulling the truck, which in turn pulled the trailer supporting the load, and a bull dozer in the rear, pushing the trailer.

In some respects, the hauling of the blades constituted the most difficult transportation problem. While the blade load, which included a portion of the hub assembly at the root of the blade, was not as heavy as the pintle girdle, tower cap or main gear box load, its great length made a careful consideration of clearance necessary, and the relatively fragile construction of the blade proper necessitated careful handling.

Fig. 4 shows a view of Grandpa's Knob at about the time the



FIG. 4.—PANORAMIC VIEW OF GRANDPA'S KNOB

heavy loads started to arrive. The wind turbine tower, which is the right hand structure of the three shown, had already been erected. To the left of the wind turbine tower is seen the erection derrick.

Fig. 5 shows a portion of these hauling operations. In this picture one of the blades is being hauled around a hairpin bend in the road built up the mountain.

Mr. S. D. Dornbirer, of the S. Morgan Smith Company, acted as Erection Superintendent for the sponsors of the project. The lifting of the various units into their proper position on the top of the tower required careful planning, but no difficulties were encountered during these operations.



FIG. 5.—HAULING THE BLADE

Fig. 6 shows the hoisting of the pintle girder. This assembly, weighing 45 tons, was the heaviest lift. On the front under side of the pintle girder may be seen an open hole, which had to be lowered on to the vertical pintle shaft which extended from the tower cap. The clearance on the shaft, which was 24 inches in diameter, was only .005 in., but no difficulty was encountered in lowering the pintle girder on the shaft, although some had been anticipated.

The raising of the blades presented the most difficult of the erection problems, since they were not only fragile, but because of their large exposed area, were extremely sensitive to wind pressure while being hoisted. With competent meteorological advice, it was possible to hoist the blades during a period of low winds.

In Fig. 7 one of the stages occurring during the hoisting of the first blade is shown. The first blade was hung on the under side of the hub assembly. This blade was then turned mechanically through 15 degrees, when it was picked up by the erection derrick and lifted to the maximum height possible—limited by the length of the derrick boom. In this latter position it was about 20 degrees above the horizontal. A temporary A-frame had been built on the side of the turbine shaft opposite the first blade. By pulling down on the temporary A-frame, the first blade was raised to the vertical position. The second blade was then hung on the under side. This was the last major step in the field erection program.



FIG. 6.—RAISING THE PINTLE GIRDER

Fig. 8 shows the completed turbine.

CONTROL HOUSE

The test unit may be operated in the following three manners:

1. Manually from aloft;
2. Manually by remote control from the control house;
3. As a completely automatic substation from the control house.

The control house, which is of substantial construction, is built into the side of the mountain, 100 yards from the main unit. This building houses the automatic switchgear and the control panels. It also houses the instrument panel on which a large number of dials



FIG. 7.—HOISTING THE FIRST BLADE

indicate the action of the unit. The indications of these dials are recorded photographically by means of two motion picture cameras. A slow speed camera takes eight pictures per minute and is in operation whenever the turbine is running. A higher speed camera takes eight pictures per second and is used only when special tests are being performed.

Photographic recording is believed, in this instance, to be more economical than recording on paper charts. It has the advantage also of leading to simultaneous readings of all the instruments. Of considerable importance is the fact that photographic recordings are more



FIG. 8.—THE COMPLETED TEST UNIT

accurate than pen recordings, since the drag of a pen on paper introduces appreciable error when sensitive readings are involved. Compared with the error due to the drag of a pen, that due to the inertia of a dial needle is small.

The power brought to the control house from the wind turbine is at 2300 volts. Just outside the control house a transformer steps this up to the 44,000 volts of the Central Vermont Public Service Corporation's high line.

ANEMOMETER MAST

Prior to the selection of Grandpa's Knob as a site for the test unit, the wind records which had been taken had as their purpose, power

output estimates. As soon as the test site was chosen, however, a program was initiated for the purpose of obtaining comprehensive data regarding wind action on mountain tops. A new anemometer mast, shown in Fig. 9, having a height of 185 feet, was built.



FIG. 9.—THE NEW ANEMOMETER MAST

One hundred twenty feet above the ground, four cross arms, each 65 feet in length, extend north, south, east and west. In this manner, three coordinate axes in space are obtained. On this mast are mounted seventeen anemometers of three different types. There are Stewart cup anemometers, for long term power output studies, and a heated rotor type of anemometer especially designed to give continuous readings for power output studies during icing conditions, when the standard

cup anemometer is unreliable or often stops completely. A third type of anemometer used is a pressure instrument, based on the pitot tube principle. This latter type will measure average velocities over a period of one second. Among the important objectives of this anemometry program, the establishment of vertical wind gradients for prevailing winds and maximum instantaneous gust gradients along each of the three coordinate axes should be mentioned. The rate of change of wind velocity during gusts is also important. Rates of change of wind direction are being studied on the basis of a recording anemoscope.

TEST PROGRAM

The test program which is now under way has the following three major objectives:

1. To find out whether the unit is satisfactory from structural, mechanical and electrical viewpoints.
2. To obtain the data which will be necessary in order to design a production unit which will be more economical than the test unit.
3. To obtain long term power output data so that the annual output of such an installation may be determined.

At the present writing the first portion of the program is essentially completed. In its basic operation the unit is satisfactory—structurally, mechanically and electrically. Quite a number of changes have been made in the unit since it was first started up and more changes will undoubtedly be made. If new units are designed, there are features which will be altered as a result of experience with the first test unit.

To carry out the second portion of the test program and obtain the data necessary for re-design, and to design a production unit on the basis of this data, and to estimate its cost, will require considerable time.

To carry out the third portion of the test program, and obtain reliable information regarding power outputs will also require considerable time, since such a study is essentially statistical in its character.

Until the cost of a production unit and the predicted power output



FIG. 10.—THE WIND TURBINE IN OPERATION

of such a unit are known with more certainty than exists at the present time, it does not seem desirable to present figures in connection with possible power costs. Before this project was sponsored, careful estimates were made regarding the cost of electric power as obtained from

wind energy. Such data as have been obtained up to this time are reasonably consistent with the original estimates, but there is not sufficient data to be conclusive.

CONCLUSION

The results of the test to this time indicate that it is possible to produce A.C. power from wind energy and they seem to justify the hope that power so produced, if used in conjunction with hydroelectric power, may prove to be economical. It should be stressed that the project is still in an experimental stage. No one realizes more keenly than those intimately associated with the project that there is still much to be learned before the economical large scale use of wind power can be accepted as a proven fact. The presentation of this paper at this time constitutes a progress report of the experiment on Grandpa's Knob. The development has been of extreme interest to those of us who have had the privilege of being connected with it, and it is hoped that this report will be of interest to the civil engineering profession.

ACKNOWLEDGMENT

The writer wishes to point out that the carrying out of this project has been made possible only by the cooperative effort of a large number of people, some of whom have been mentioned in this paper. Space does not permit listing the names of all the key men in this project but particular mention should be made of Mr. Beauchamp E. Smith, Vice President and General Manager of the S. Morgan Smith Co., Mr. Burwell B. Smith, Vice President and Secretary of the S. Morgan Smith Co., Mr. Palmer C. Putnam, inventor of the turbine, Mr. George A. Jessop, Chief Engineer of S. Morgan Smith Co., Mr. Thomas S. Knight, Vice President of the General Electric Company, Mr. George Jump and Mr. William Bagley, of the General Electric Company, and Mr. Harold L. Durgin, Chief Engineer of the Central Vermont Public Service Corporation.

The writer wishes to express also his personal appreciation for the able assistance given him by Professor Charles H. Norris, of the Civil Engineering Department of M. I. T., in connection with the many complex structural problems encountered, and to Mr. Grant H. Voaden of the S. Morgan Smith Company, who served as Assistant Chief Engineer of the Project.

OPERATION OF THE WARDS ISLAND SEWAGE TREATMENT PLANT

BY GAIL P. EDWARDS*

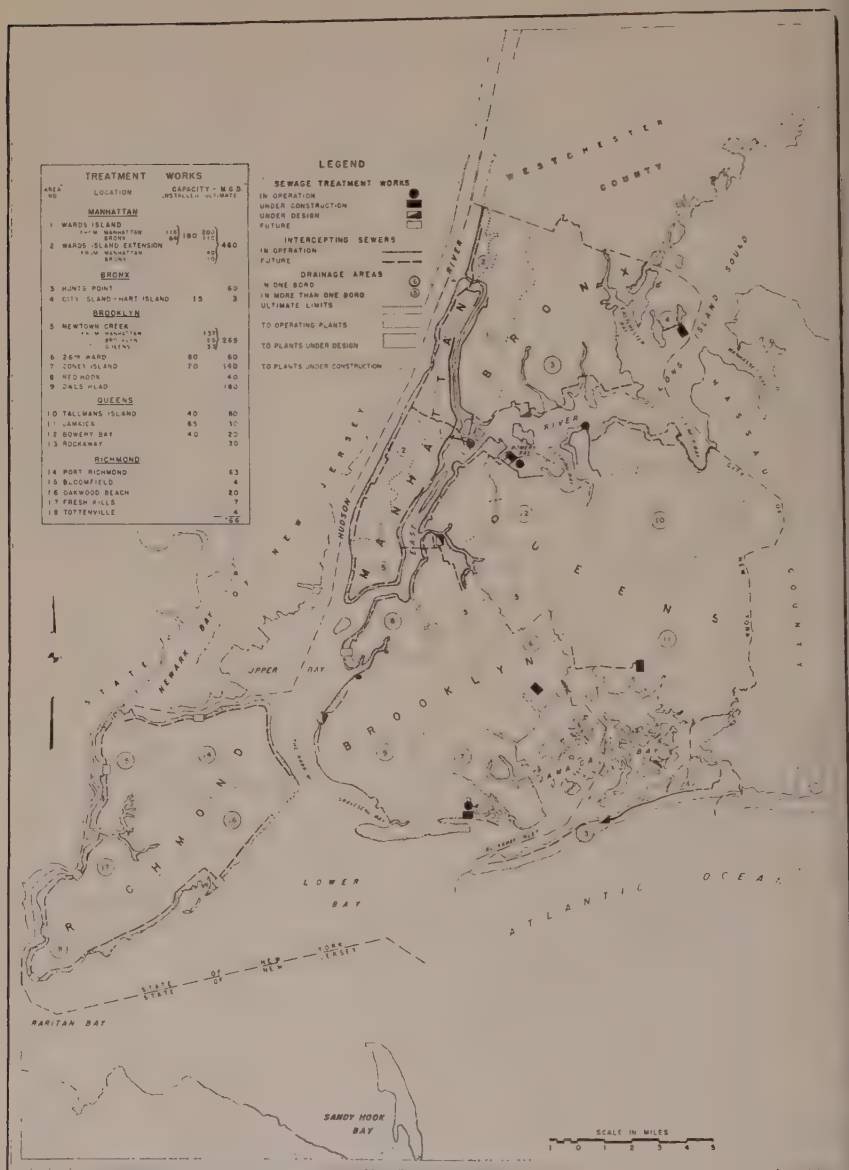
(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers on January 7, 1942)

THE Wards Island Activated Sludge Plant, the largest in New York City is located on Wards Island in the East River on that section east of the Hell Gate Bridge of the New York Connecting Railroad. Construction work on the dock, sea wall, tanks and operating galleries was started in June, 1931 and completed in 1933 at a cost of \$4,500,000. In 1935 the city secured a P. W. A. grant of 45 percent amounting to about \$11,000,000 for construction of the intercepting sewers, grit chambers, tunnels, superstructure and machinery. The plant which was put into service in October, 1937, was designed for a sewage flow of 180 m.g.d. It serves a population of about 3,300,000 living in an area of about 10,600 acres. About one-third of the area and slightly more than one-half the people served are in Manhattan. The remainder are in the Bronx.

The flow of sewage in the interceptors is controlled by regulators, 2 in Manhattan and 8 in the Bronx, which are designed to reject the amount in excess of twice the design flow. In the regulators, a float activates a valve which permits the excess sewage to flow to the river. Tide gates prevent the flow of river water into the intercepting sewers.

Two grit chambers, similar in design are located one in Manhattan and one in the Bronx. Floating material is removed by racks and bar screens with openings 3" and 1" respectively. The coarse screenings are removed by the Dept. of Sanitation and the finer solids are ground and returned to the sewage. Each grit chamber has four channels about 50 feet long so designed that the velocity of flow is maintained between 0.8 and 1.2 feet per second. The grit is washed, removed by spiral conveyors to ejectors and lifted to elevated storage tanks from which it is removed by the Dept. of Sanitation to incinerators.

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JUNE 1901. PREPARED BY THE GEOGRAPHICAL SECTION

FIG. 1.—TENTATIVE PLAN FOR SEWAGE DISPOSAL UNDER THE JURISDICTION OF THE DEPARTMENT OF PUBLIC WORKS, CITY OF NEW YORK

The sewage flows from the grit chambers to Wards Island through tunnels in rock. The tunnel from the Bronx has an inside diameter of 10.5 feet and at its deepest point is 150 feet below the Harlem River. It is about a mile in length. The Manhattan tunnel has an inside diameter of 8.5 feet and because of a fault in the rock formation is 520 feet below the East River at its deepest point.

Sewage from both tunnels rises in the shafts at Wards Island and passes through a surge chamber to the suction well of the pumps. Six raw sewage pumps, each with a capacity of 90 m.g.d. are provided to lift the sewage so that it may pass through the remainder of the plant by gravity.

Eight cross flow primary settling tanks, each 100 feet square with an effective depth of 15 feet provide a detention period of about one hour at a flow of 180 m.g.d.

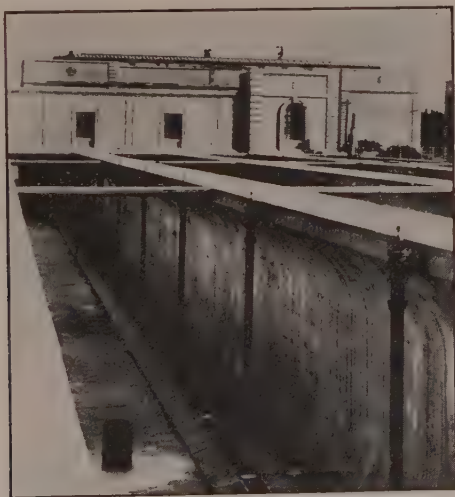


FIG. 2.—INTERIOR—AERATION TANK, WARDS ISLAND SEWAGE TREATMENT PLANT

The sixteen aeration tanks are arranged in four batteries of four tanks. Each tank has four passes which are about 345 feet long, 22 feet wide and 15 feet deep. Air is added through two rows of diffuser plates placed at one side of each pass to give a spiral flow. At the design flow and with 25% return sludge, the detention period is 5.25 hours.

The aeration liquor flows to the 32 rectangular conveyor type, final settling tanks which are 12 feet deep and have an area of 7500 sq. ft. The detention period at design flow is about 3 hours. The sewage is added at the center and flows to weirs about 10 feet from the end wall. New cross troughs and weirs have been added to seven tanks of one battery to improve settling. Four of the final settling tanks, one in each battery, are used for concentrating the excess activated sludge. In these tanks, the average detention period is about 9 hours and the concentration of solids in the sludge usually varies between 2 and 3 percent. The effluent from the plant is discharged into the East River through two outfalls located at either end of the dock.

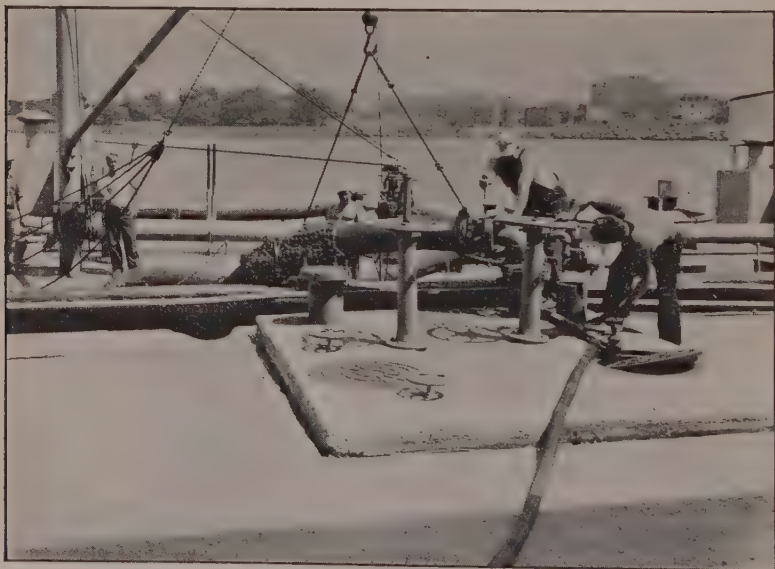


FIG 3.—LOADING A SLUDGE VESSEL AT WARDS ISLAND SEWAGE TREATMENT PLANT

The primary tank and concentrated excess activated sludge are pumped to the sludge storage building located at the dock. This building contains four steel storage tanks each with a capacity of 47,500 cu. ft. Two of these are used for primary sludge and two for excess activated. Advantage can be taken of settling which occurs in

these tanks by returning the supernatant liquor to the raw sewage pumps. Sludge flows from the tanks by gravity through a 12" cast iron line with flexible hose connections to the sludge vessels. Three ocean-going vessels, identical in design, are equipped with twin Diesel engines and are about 257 feet long and 44 feet wide. They have a speed of about 10 knots and a capacity of 55,000 cu. ft. A vessel can be loaded in about 90 minutes and as the sludge is stored above the water line, (Fig. 4) it can be unloaded in 5 to 7 minutes by

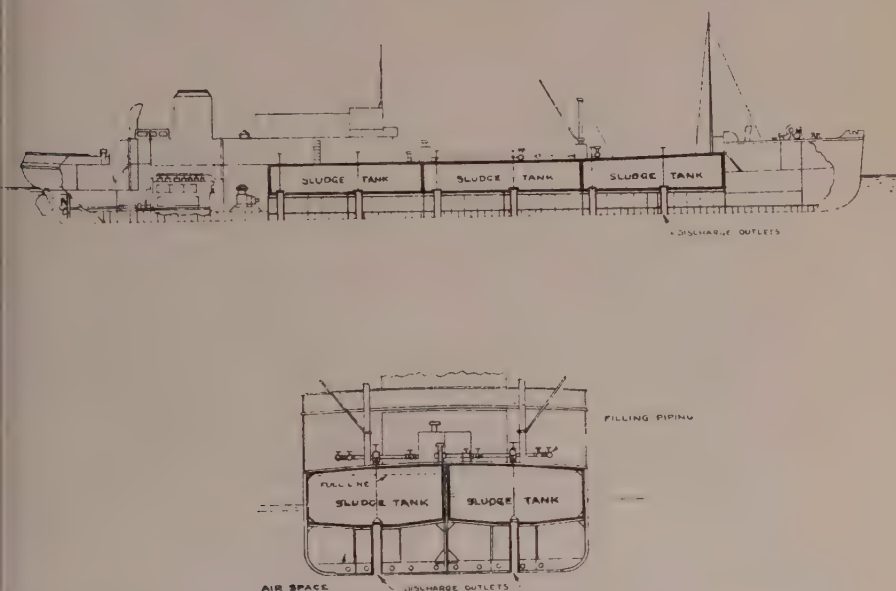


FIG 4.—LONGITUDINAL AND CROSS SECTIONS OF SLUDGE VESSEL, SHOWING SLUDGE TANKS AND DISCHARGE OUTLETS

gravity. The dumping ground is about 34 miles by water from the plant at a point about 12 miles from the Long Island and New Jersey shores. A round trip requires about 6.5 hours. Permits for each boat to dump sludge at sea are obtained from the Supervisor of the Harbor twice a month. After each unloading, a certificate must be surrendered to a patrol boat of the Supervisor of the Harbor. Two vessels are kept in active service and have averaged about three trips a day. One vessel is tied up for repairs and maintenance by the crew

WARD'S ISLAND SEWAGE TREATMENT WORKS
OPERATING SUMMARY 1940

M O N T H	SEWAGE				TREATMENT				REMOVED FROM SEWAGE											
	TREATED		% RETURN	AIR CU FT	SUSP. SOLIDS AER. PPM.	SLUDGE		RETENTION PERIOD. HRS	FINAL TANKS SQ. FT.	POWER K.W.H. PER DAY	RAKINGS		GRIT		TOTAL SLUDGE TO SEA THOUS. CU FT					
	PRIM (1)	AER (2)				SLUDGE (3)	PER (4)				WET TONS (12)	EST. CU FT X TOTAL (13)	PER MG (14)	CUBIC FEET (15)		PER MG (16)				
	JAN.	176	174	38	0.75	2000	0.66	5.3	3.2	716	121,000	703	167	6664	1.2	12,339	2.4	5580	4916	
FEB.	176	173	50	0.77	1950	0.65	4.9	3.2	710	126,000	714	196	7840	1.5	12,358	2.4	5008	4576		
MAR.	181	192	47	0.65	2180	0.55	4.7	3.0	748	125,000	653	156	6240	1.1	16,452	2.8	4853	5184		
APR.	206	191	44	0.58	1825	0.65	4.6	2.9	785	115,000	558	152	6776	1.0	22,955	3.7	3561	5052		
MAY	194	192	46	0.57	1850	0.69	4.5	2.9	789	113,000	584	144	5776	1.0	17,733	3.0	3483	5104		
JUN.	203	203	41	0.60	1750	1.03	4.1	2.5	899	122,000	601	99	3956	0.7	14,789	2.2	3476	5108		
JUL.	205	205	41	0.62	1750	0.79	4.4	2.6	874	122,000	595	111	4432	0.7	14,427	2.3	3322	4383		
AUG.	192	192	47	0.64	1525	1.13	4.2	2.6	810	118,000	609	115	3722	0.6	11,561	1.9	3520	4697		
SEP.	195	195	45	0.70	1525	0.75	4.4	2.6	868	125,000	631	112	4488	0.8	10,832	1.9	3498	4625		
OCT.	199	194	47	0.70	1725	0.66	4.5	2.8	823	125,000	644	113	4536	0.7	11,217	1.8	3446	4522		
NOR.	199	199	42	0.63	2300	0.98	4.4	2.8	847	123,000	612	124	4928	0.8	10,402	1.7	3460	4633		
DEC.	178	178	49	0.62	2325	0.87	4.6	3.0	769	115,000	615	151	6045	1.1	7,482	1.4	3356	4071		
AVE.	193	190	44	0.65	1990	0.78	4.6	2.9	803	120,000	625	135	5394	0.9	13,600	2.3	3881	4739		
										TOTAL		1618		64,725		163,147		46,572		56871
Estimated at 50 lbs per cu ft																				

* Estimated at 50 lbs. per cu ft

TABLE 2
WARD'S ISLAND SEWAGE TREATMENT WORKS
PLANT EFFICIENCY—SUMMARY 1940

Month	SUSPENDED SOLIDS—P.P.M.										5 DAY 20° C.—B. O. D.					Sludge To Sea % Sol.	
	Raw Sewage	Prim. Effl.	% Rem.		Aer. Eff.	Plant Eff.	% Rem.	Raw Sewage	Prim. Effl.	% Rem.	Aer. Eff.	Plant Eff.	% Rem.	% Sol.	(15)		
			(1)	(2)												(3)	(4)
Jan.	200	121	40	23	89	24	88	251	190	24	18	93	20	92	2.81*		
Feb.	198	116	41	18	91	20	90	235	174	26	12	95	15	94	2.87*		
Mar.	260	121	53	15	94	20	92	247	167	32	11	96	19	92	3.39*		
April	231	146	37	15	94	21	91	227	169	26	12	95	23	90	4.52*		
May	250	148	41	13	95	15	94	225	160	29	12	95	14	94	4.65*		
June	239	136	43	14	94	14	94	209	151	28	11	95	11	95	4.66*		
July	220	188	25	12	95	12	95	190	158	17	10	95	10	95	4.19*		
Aug.	175	148	15	11	94	11	94	160	139	13	9	94	9	94	4.24*		
Sept.	198	165	—	13	94	13	94	172	159	—	12	93	12	93	4.20*		
Oct.	200	175	—	12	94	17	91	179	168	—	15	92	19	89	4.17*		
Nov.	207	235	—	11	95	11	95	177	191	—	10	94	10	94	4.25*		
Dec.	259	239	—	14	95	14	95	211	194	—	9	96	9	96	3.85*		
Ave.	220	161	—	14	94	16	93	207	168	—	12	94	14	93	3.88*		

*Weighted average.

and for an annual overhauling. In 1940, the ships made 932 trips covering about 63,000 miles. An average of 155 dry tons of sludge was removed daily.

The operating summary and record of plant efficiency for 1940 are shown in Tables 1 and 2. The ratio of return sludge solids to the suspended solids in the primary tank effluent is 14.3 and 511 cubic feet of air were used to remove one pound of five day B. O. D. in the aeration tank. An average of 4.1 p.p.m. of dissolved oxygen was maintained in the effluent from the aeration tanks, 3.5 p.p.m. in the effluent from the final tanks and 6.5 p.p.m. in the activated sludge effluent as it was discharged from the plant.

The pH of the raw sewage usually varies between 6.8 and 7.2. Occasionally because of industrial wastes, readings as low as 4.8 and as high as 8.5 are obtained. The chlorides average 300-400 p.p.m. but run as high as 2800 in the Manhattan sewage. High concentration of chlorides indicates leakage of river water into the interceptors through the tide gates.

The area tributary to the Wards Island Plant is densely populated and contains many industrial establishments. The results of a preliminary survey indicate that the industrial wastes discharged to the plant have a population equivalent of about 350,000. Of the various industries, two—breweries and laundries are responsible for about 68% of the waste flow, 82% of the waste suspended solids and 86% of the waste 5 day B. O. D. and population equivalent.

Some difficulty has been encountered with diffuser plate clogging since the plant went into operation in October, 1937. Plates in 8 tanks were cleaned in 1939, in 14 in 1940 and in all 16 in 1941. Since the plates are set in concrete, they must be cleaned in place. After some experiments with chlorine and acid, a solution of 25% caustic was found to be most satisfactory. In practice, the tanks are drained, the plates are flushed with a hose, scrubbed with deck brushes and allowed to dry. Then the caustic soda solution is applied and the tanks are allowed to stand 24 hours, before returning them to service. The first pass receives two applications of caustic. The caustic is not washed out. When plates are clean, the air pressure is about 7 pounds, 6 ounces per square inch. This pressure gradually builds up to 8 pounds, 9 ounces and at this or higher pressure, some plates may blow out.

Chlorine has been added to the return sludge periodically during the past two or three years. Chlorine seems to aid in the control of bulking and intermittent dosing is useful. There is some question as to whether full value is received in continuous applications.

It is of fundamental importance in the operation of a sewage treatment plant to avoid unsightliness and disagreeable odors. This is particularly true when the plant is located near residential property or densely populated areas. The Wards Island plant located under the Triboro Bridge is exposed to the view of thousands of people who pass over the bridge daily. A portion of the residential section of Astoria is located immediately across the East River, not more than a quarter mile away.

Waste sludge is stored in large tanks at the plant in a building on the waterfront before it can be taken to sea. The storage of sludge and the loading of the sludge vessels especially in the summer are potential sources of nuisance.

Odors arising from the Sludge Storage Building are successfully controlled with the aid of ozone and activated carbon. Foul air around the tanks is drawn by a fan (15,000 c.f.m.) through cannisters of activated carbon, then dosed with ozone before discharge at the roof of the building. The ozone equipment consisting of two batteries of generators and one silica gel dehydrator for drying air used in the generators, has a capacity of 61 grams per hour. The generator consists essentially of a glass cylinder with a copper coating on the inside and an aluminum cylinder around it with an air space between. Electrical connections are made to the copper coating and the aluminum cylinder. The current passes through the glass and breaks down the dry air fed through the annular space. Although a high voltage (9,000) is required, the cost of electricity for ozone production is only about 56 cents per day.

The odors arising from the loading of the sludge vessels are taken care of with cannisters of activated carbon fitted into gas vents of the sludge storage compartments. The main difficulty with the carbon is its low capacity for odor removal which necessitates frequent replacement.

The Wards Island plant was built to relieve pollution in the river and harbor waters. The effect on the Harlem River is most pro-

nounced because much of the sewage which is now treated at the plant previously entered the Harlem River below 190th Street in Manhattan and Jerome Avenue in the Bronx. A study of the condition of the waters in the rivers and harbor about New York City has been made

TABLE 3
AVERAGE PERCENTAGE OF SATURATION OF DISSOLVED OXYGEN IN MAIN
BRANCHES OF NEW YORK HARBOR, JUNE 1ST TO OCTOBER 1ST

Year	Samples	Hudson River Below Spuyten Duyvil	Harlem River	Upper East River	Lower East River	Upper Bay	Kill Van Kull	Narrows	Arthur Kill	Jamaica Bay
1909	404	72	55	86	65	67	79	83		
1911	861	62	42	69	54	72	70	76		
1912	150	58	—	—	49	64	65	71		
1913	880	57	29	—	43	66	65	69		
1914	473	50	30	50	40	71	—	68		
1915	245	43	28	—	33	72	—	78		
1916	176	46	24	—	26	64	—	63		
1917	238	42	22	47	29	50	—	63		
1918	54	54	23	50	21	56	—	61		
1919	320	36	29	30	24	51	35	58		
1920	264	44	23	50	27	43	42	52		
1921	258	30	15	37	16	33	38	35		
1922	280	44	26	51	26	51	51	60		
1923	354	37	27	38	22	47	43	57		
1924	643	44	26	45	26	48	43	73		
1925	662	41	27	50	26	46	47	55		
1926	396	23	14	37	13	26	29	40	26	66
1927	368	35	17	40	21	27	34	48	50	59
1928	433	37	28	41	23	47	41	44	56	69
1929	382	47	30	62	29	45	41	52	85	71
1930	332	37	25	40	20	38	39	51	49	71
1931	532	47	27	50	23	37	42	48	45	70
1932	582	41	20	38	16	43	47	51	71	71
1933	568	44	22	41	20	40	46	51	44	80
1934	751	47	22	39	16	39	50	52	55	69
1935	705	40	20	38	15	39	44	52	58	61
1936	520	36	14	42	15	38	41	51	47	61
1937	706	46	24	42	22	41	47	48	55	71
1938	524	46	31	24	22	39	41	54	45	61
1939	647	46	32	47	25	45	41	53	45	71
1940	742	46	32	41	23	43	48	58	56	71
1941	602	46	29	41	21	36	35	47	47	71

since 1909. In 1938, the year after the Wards Island plant was put into operation, the average percent saturation of dissolved oxygen in the entire Harlem River was 31, the best value since 1911. For the two points most directly affected, 155th Street and Willis Avenue the average for 1938 was 22% or nearly twice the average for the previous eight years. For the first time since 1927, no zero dissolved oxygen was found in the Harlem River. This improvement has been maintained until this last summer when because of dredging operations, zero dissolved oxygen was found. A summary of the harbor results is shown in Table 3.

The cost of maintenance and operation has shown a pleasing tendency downward. The overall costs have been as follows:

1938	\$11.86 per million gallons
1939	11.34 per million gallons
1940	11.29 per million gallons

These figures are made up, approximately, of salaries and wages, 45-50%, power and light, 40% and supplies, repairs and miscellaneous 10-15%

The operation and maintenance cost of disposal of sludge to sea which is included in the overall cost was \$2.14 per million gallons of sewage treated in 1938, \$2.07 in 1939 and \$1.88 in 1940. Based on dry tons, the cost was \$3.28 in 1938, \$2.51 in 1939 and \$2.40 in 1940. It should be noted that in 1938, only sludge from Wards Island was handled. In 1939, a small amount of sludge from the Bowery Bay plant was included and in 1940, sludge from Wards Island, Bowery Bay and Tallmans Island plant was handled.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

APRIL 15, 1942.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club.

This was a Joint Meeting with the Northeastern Section, American Society of Civil Engineers, Prof. Harry P. Burden, presiding for that Society. Sixty-five members and guests attended; fifty-eight persons attended the supper.

Vice-President Athole B. Edwards, presiding for the Boston Society of Civil Engineers announced the death of the following members:

Franklin A. Snow, who had been a member since March 21, 1894, and who died March 19, 1942.

William F. Cummings, who had been a member since May 17, 1929, and who died April 10, 1942.

Prof. Harry P. Burden introduced the Speaker of the evening, Dr. Harold E. Wessman, New York University, who gave a very interesting paper on "The Technical Aspects of Air Raid Protection".

The meeting adjourned at 8:45 P.M.

EVERETT N. HUTCHINS, *Secretary*

MAY 20, 1942.—A regular meeting of the Boston Society of Civil Engi-

neers was held this evening at the Engineers Club, and was called to order at 7:00 P.M., by Vice-President Athole B. Edwards. Fifty members and guests attended this meeting. Forty-six persons attended the dinner.

The Secretary reported on the election of the following new members:

Grade of Member: David M. Brown, H. Lowell Crocker, Chester H. Minehan, George T. Rooney, Erik Per Sorensen, Norman P. Spoiford.

Grade of Junior: Paul C. Danforth,* Walter B. Kelley,* Arthur M. O'Connor, Jr.*

Grade of Student: Anthony J. White.

The Vice-President then introduced the speaker of the evening, Dr. Karl Terzaghi, Lecturer on Soil Mechanics and Engineering Geology at Harvard University, who gave an interesting talk on "Shield Tunnels of the Chicago Subway." The talk was illustrated.

Meeting adjourned at 9 P.M.

EVERETT N. HUTCHINS, *Secretary*

DESIGNERS' SECTION

APRIL 8, 1942.—The April meeting of the Designers' Section was held in the Society's rooms. It was called to order by Chairman Emil A. Gramstorff at 6:50 P.M.

*Transfer from Grade of Student.

The clerk's report of the annual meeting held March 11, 1942, was read and accepted. The chairman then called attention to the Conference on Aerial Bombardment Protection to be held April 18, 19 and 20, 1942, at Northeastern University. Prof. Albert Haertlein announced that forms were available for the Registration of Professional Engineers and Land Surveyors for the Commonwealth of Massachusetts.

The chairman then introduced Mr. La Motte Grover, Structural Welding Engineer, Air Reduction Sales Company, New York, N. Y. Mr. Grover spoke on the subject "Welded Steel Structures—Design and Construction." His talk had particular reference to structural welding. Some points which he emphasized as important in obtaining good welding were control of welding procedure, qualification of welding operators and provision in design of the structure of proper layout for welding details. A question period followed Mr. Grover's talk.

Thirty-five members and guests were present. The meeting was adjourned at 8:50 P.M.

L. M. GENTLEMAN, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[June 20, 1942]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional repu-

tation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

JOHN A. MCAULIFF, Newton Highlands, Mass. (b. November 26, 1890, Providence, R. I.) Graduated from Tufts College of Engineering, 1913, with B.S. degree in structural engineering. Experience, in the fall of 1913, with Mead-Morrison, as inspector of coal handling machinery; January, 1914, with Henry A. Wentworth, Consulting Engineer, acting as assistant and office manager, also did research work mostly connected with mining projects and developments; April, 1915, with Stone & Webster Engineering Corporation as purchasing agent; in the Fall of 1917, transferred to the Hog Island Shipbuilding Plant at Hog Island, Philadelphia; resigned from Stone & Webster Corporation in the Fall of 1919 to enter own business of designing and selling equipment and also supervise installation of private and semi-public sewage disposal plants, for camps, factories, schools, institutions of all types, etc. Have been engaged in nothing but engineering work, or in sale of steel products used entirely in the engineering field and requiring engineering experience. Refers to J. S. Crandall, A. E. Harding, M. Linenthal, F. N. Weaver.

CARL H. SELLS, Quincy, Mass. (b. July 2, 1891, South Boston, Mass.) Graduated from Harvard Grammar School, Cambridge, Mass., in 1906;

Rindge Manual Training High School, Cambridge, Mass., three and one half years; Massachusetts State Nautical School, six months; Franklin Union, four years of evening courses in advanced mathematics and engineering; graduated and received commission as Ensign in the United States Naval Reserve Force during World War from First Naval District Ensign School at Harvard University; Pace and Pace School of Law and Accounting four semesters, two years of evening courses; State University Extension, three years evening courses in engineering; State University Extension, five years evening miscellaneous courses. Experience, February, 1910, to October, 1917, with Boston Elevated Railway Company, as Rodman and Transitman; leave of absence October, 1917, to enter United States Naval Reserve Force during World War until March, 1919; March, 1919 to May, 1920, Boston Elevated Railway Company as transitman and draughtsman; May, 1920, to March, 1933, Boston Elevated Railway Company, assistant engineer; March, 1923, to February, 1924, Domestic Oil Burner Company as sales engineer; February, 1924, to July, 1925, Hugh Nawn Contracting Company as estimator and engineer; July, 1925, to present date with Boston Elevated Railway Company as Assistant Engineer. Refers to *F. C. Bell, A. J. Blackburn, H. M. Steward, E. L. Lockman.*

PERE O. HAAK, Cambridge, Mass. (b. May 15, 1911, Stockholm, Sweden.) Graduated from the Rindge Technical School in Cambridge, in 1929 and in 1934 completed a four year course in Structural Design at the Franklin Technical Institute in Boston; in 1935 completed a two year course in Structural Design at Lowell Institute (M. I. T.), Cambridge, Mass. Experience, 1930 to 1933, employed as a draftsman in the maintenance division of the George

Steck Piano Company, Neponset, Mass., duties consisting of general plant layouts, the design and detail of light steel frame buildings for the storage and drying of lumber, etc.; 1934-1935, employed by L. M. Hersum, Consulting Engineer, Boston, Mass., on general structural design and drafting; 1935, employed by the F.E.R.A. as project Engineer, duties consisting of design of sewers and general municipal construction projects under the supervision of Edgar Davis, Deputy Engineer, Cambridge, Mass.; 1936 to the end of 1941, employed in the Engineering Department of Warren Brothers Roads Company, under the supervision of R. C. Paine, Chief Engineer, duties consisted of the development and the structural and mechanical design connected with Asphalt Paving Plants, Stone Crushing Plants, Gravel Plants and Material Handling equipment. At present am employed as a Structural Designer by the E. B. Badger Company, 75 Pitts Street, Boston, Mass. Engaged chiefly in Reinforced Concrete Design as applied to the engineering of Oil Refineries, Chemical Plants and Synthetic Rubber Plants. Refers to *L. S. Burke, W. H. Blank, E. W. Davis, C. W. Newcomb.*

JOHN E. BAMBER, Dedham, Mass. (b. December 27, 1918, Norwood, Mass.) Graduated from Norwood High School in 1936; received B.S. Degree in Civil Engineering from Northeastern University in 1941. Experience, October, 1938 to April, 1939, transitman with Aspinwall & Lincoln, Boston, Mass.; September, 1939 to April, 1941, instrumentman and draftsman with the New York, New Haven & Hartford Railroad Company; June, 1941 to present date with U. S. Engineer Department, Boston, Mass., as Engineering Aide. At present Senior Engineering Aide in charge of layout at Norwood Airport, Norwood, Mass. Refers to

F. O. Baird, A. E. Everett, E. A. Gramstorff, L. W. Lenfest.

For Transfer

PAUL M. LEVENSON, Newport, R. I. (b. February 25, 1917, Boston, Mass.) Graduate of Northeastern University School of Engineering, June, 1940, U.S. Degree in Civil Engineering. Experience, 1936, rodman and draftsman, New England Survey Service, Boston, Mass.; 1937, rodman and draftsman, C. W. Branch, Quincy, Mass., 1938, Junior Engineering Aide, Metropolitan District Commission, Sewer Department; 1940-1942, Engineer for Platt Contracting Company, Inc., Newport, Rhode Island. At present Engineering Coördinator for Platt Contracting Company, Inc. Refers to *C. O. Baird, E. A. Gramstorff, R. S. Slayter, A. E. Everett.*

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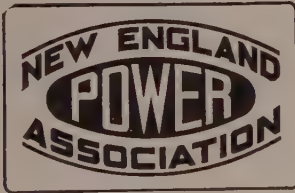
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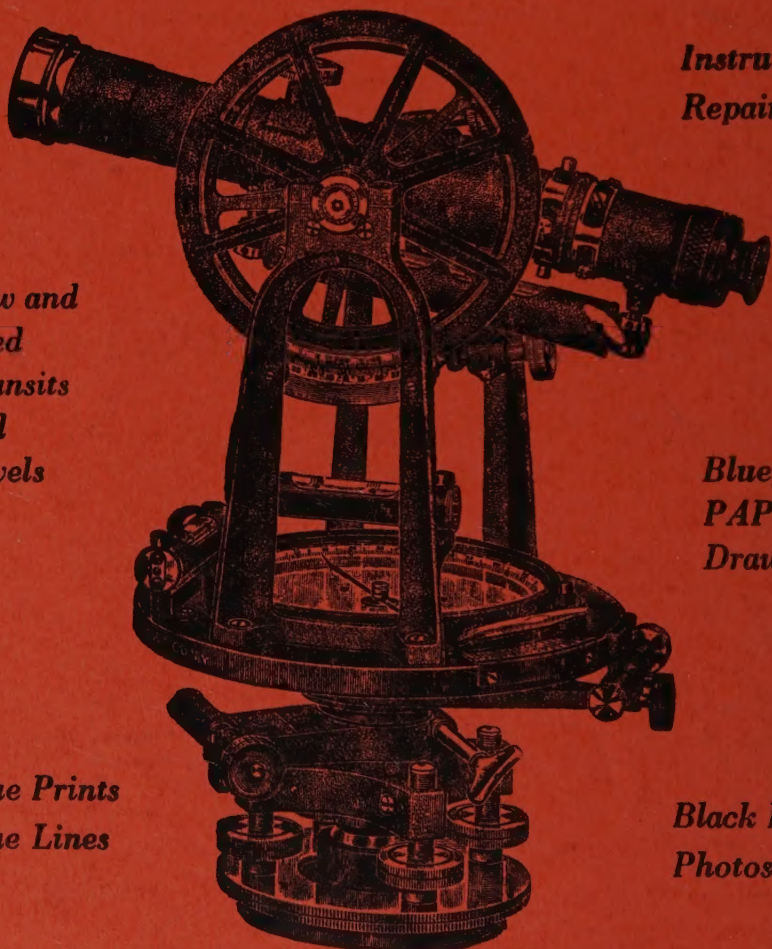
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